EXPERIMENTAL AND THEORETICAL STUDY INTO ON BOTTOM STABILITY OF SUBMARINE PIPELINE UNDER WAVE AND CURRENTS

AZAM SHAUQI BIN ABDUL HALIM

CIVIL ENGINEERING

UNIVERSITI TEKNOLOGI PETRONAS

JANUARY 2009

Experimental and Theoretical Study Into On Bottom Stability of Submarine Pipeline under Wave and Currents

by

Azam Shauqi Bin Abdul Halim

Dissertation submitted in partial fulfilment of

the requirements for the

Bachelor of Engineering (Hons)

(Civil Engineering)

JANUARY 2009

Universiti Teknologi PETRONAS
Bandar Seri Iskandar
31750 Tronoh
Perak Darul Ridzuan

CERTIFICATION OF APPROVAL

Experimental and Theoretical Study Into On Bottom Stability of Submarine Pipeline under Wave and Currents

by

Azam Shauqi Bin Abdul Halim

A project dissertation submitted to the
Civil Engineering Programme
Universiti Teknologi PETRONAS
in partial fulfilment of the requirement for the
BACHELOR OF ENGINEERING (Hons)
(CIVIL ENGINEERING)

Approved by,

(Associate Professor Dr. Saied Saiedi)

UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

January 2009

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

AZAM SHAUQI BIN ABDUL HALIM

ABSTRACT

Pipelines are one of the methods that widely used in oil and gas industries to transport their products as it is economically efficient. Besides that, it is also proven that submarine pipeline is friendly towards the environment as it only contributes 2 percent from total oil spills in the ocean. As it is name, submarine pipeline is lay under the ocean which are hostile environment toward us. Continuously exposed to the environmental load such and wave and currents, these pipelines need to be stable so that it won't get fatigue due to constant movement. Therefore as the On Bottom Stability is vital towards submarine pipeline, this project is concentration on the experimental and theoretical study regarding these aspects. It consists of two parts which are experimental study to measure the force acting on the model pipeline and also to observe the behavior of the pipeline under the acting force. The other one is study using the DnV spreadsheet to observe how the pipeline stability varies with the parameters regarding it. For the experimental part, there are mechanism that being design to measure the force acting on the pipeline model due to the wave and current and result from this experiment then will be compare with result from the industrial spreadsheet.

ACKNOWLEDGMENT

First and foremost, I give thanks and praise to God for His guidance and blessings throughout the entire course of my training.

I wish to express my gratitude to AP Dr Saied Saiedi, for his guidance and support throughout completing my Final Year Project in the Universiti Technologi Petronas Civil Engineering Department. His knowledge is vital for the projects and activities that I have involved.

Many figures had provided immeasurable amount of guidance, ideas, assistance, support and advice. Without help from these people, this Final Year Project may not be that meaningful and successful. Greatest appreciation is expressed to the following individuals, Mr Meor, Mr Idris, Miss Nurul Husna Baharuddin, Miss Najwa Hamidi, Mr Hafiz, Miss Ainor Syafarizan, Ms Nabila, Mr Kalai and also to all staffs of Civil Engineering Department.

My acknowledgement would be incomplete without giving credit to Universiti Teknologi PETRONAS, especially Civil Engineering Department which has equipped students with essential skills for self-learning. Its well-rounded graduate philosophy has proven to be useful in the industry

Finally, I would like to thank my family. They have been a wonderful source of encouragement and joy to me and also not to forget the fellow students who's their help was a tremendous gift to me. May God bless all of us and only HE, the Almighty could repay all my debts to them.

ABBREVIATIONS AND NOMENCLATURES

The following definitions refer to abbreviations used throughout this document:

- ASME: American Society of Mechanical Engineers
- ANSI: American National Standard Institute
- BCOT: Bintulu Crude Oil Terminal
- CD: Chart Datum
- DD: Development Division
- DFE: Facilities Engineering Department
- DHSE: Corporate Health Safety and Environment Department
- DnV: Det Norske Veritas
- DSCM: Supply Chain Management Department
- EQR: Environmental Quality Regulation
- FWS: Full Well Stream
- HAT: Highest Astronomical Tide
- LAT: Lowest Astronomical Tide
- MAOP: Maximum Allowable Operating Pressure
- MSL: Mean Sea Level
- PCSB: Petronas Carigali Sdn Bhd
- PL: Pipeline
- SKO: Sarawak Operations
- SMYS: Specified Minimum Yield Strength
- TBA: To be advised
- TBC: To be confirmed
- TEP-A: Temana Production-A Platform
- TEP-B: Temana Production-B Platform
- TEDP-B: Temana Drilling Production-B Platform
- TEDP-E: Temana Drilling Production-E Platform
- TEJT-C: Temana Jacket-C Platform
- TEJT-T: Temana Jacket-T Platform

TEJT-W: Temana Jacket-W Platform

TEJT-CC: Temana Jacket-CC Platform

TEV-A: Temana Vent-A Platform

TEV-B: Temana Vent-B Platform

Zone2: Portion of the pipeline system located at an offshore platform from pig trap down to riser bottom bend at the seabed including an extra length of pipe of at least five pipe diameters beyond the bottom bend or fitting.

Zone 1: The remainder of the pipeline system.

The nomenclatures to be used throughout the engineering design are summarized below:

a	=	water particle acceleration normal to the pipe axis (m/s^2)
A_H	=	actual contact area between pipe and soil along unit length of pipe (m^2/m)
A_i	=	pipe flow cross-sectional area (m^2)
A_s	=	pipe steel cross-sectional area (m^2)
c	=	soil cohesion (N/m^2)
C_D	=	coefficient of drag (dimensionless)
C_I	=	coefficient of inertia (dimensionless)
C_L	=	coefficient of lift (dimensionless)
C_T	=	tension / suction strength typically ranging from 50 to 100% of the soil's cohesion depending upon the extent of sinkage i.e. effective contact area (kN/m^2)
C_1	=	numerical constant dependent on end conditions
C_2	=	Constant dependent on end conditions
D_F	=	Design Factor {allowable stress factor} (dimensionless) (i.e. 0.72 for Zone 1 and 0.5 for Zone 2) [See Ref.1].
D	=	nominal pipe outside diameter (m)
D_i	=	internal pipe diameter (m)

D_t = overall diameter of pipe including coatings (m)

D_{max} = Maximum steel pipe outside diameter (mm)

D_{min} = Minimum steel pipe outside diameter (mm)

E = Young's Modulus for steel (Pa)

E_C = Euler buckling load (N)

E_c = consumption rate of anodes (kg/A Year)

E_w = weld joint factor

F_{ai} = axial force in pipe (N)

F_D = hydrodynamic drag force (N)

F_I = hydrodynamic inertial force (N)

F_L = hydrodynamic lift force (N)

F_s = shear force applied to the pipeline (N)

F_{SLC} = lateral soil friction coefficient for cohesive soil (dimensionless)

f_n = natural frequency of pipe (Hz)

f_o = out of roundness of pipe (%)

f_v = vortex shedding frequency (Hz)

g = gravitational acceleration (m/s^2)

H_{max} = maximum wave height (m)

H_s = significant wave height (m)

h_{tr} = depth of overburden above top of pipe (m)

h = water depth (m)

I = pipe second moment of area (m^4)

I_c = output current capacity for anodes (amps)

I_m = mean current requirements for anodes (amps)

K_s = stability parameter (dimensionless)

L = length of free span (m)

L_{AN} = effective life of anode (Years)

L_o = length of Out-of-Straightness (m)

L_{CA} = pipe sinkage contact width (m)

L_{DPS} = depth of pipe sinkage (m)

M_{bi} = bending moment in pipe (Nm)

m_e = effective mass per unit length (kg/m)

m_p = mass of pipe in air per unit length (kg/m)

m_c = mass of contents per unit length (kg/m)

m_a = added mass per unit length (kg/m)

n = inner bend radius divided by pipe diameter (dimensionless)

n_i = number of stress cycles in stress block (dimensionless)

N_c = bearing capacity factor (cohesion)

N_q = bearing capacity factor (consolidation)

N_γ = bearing capacity factor (internal friction)

N_i = no. of cycles to failure at constant stress range (dimensionless)

N_x = no. of cycles to failure at a cyclic stress amplitude S_f (dimensionless)

P_{ar} = propagation pressure of buckle arrestor (Pa)

P_c = external collapse pressure (Pa)

P_e = hydrostatic pressure (Pa)

P_o = operating pressure (Pa)

P_I = design internal pressure (Pa)

P_{pr} = buckle propagation pressure (Pa)

P_y = yield pressure (Pa)

q = equivalent effective surcharge pressure as a result of overburden (kN/m^2)

q_{ULTc} = ultimate bearing resistance for cohesive soils (kN/m^2)

R = radius of curvature for pipeline bend (m)

R_a = Total circuit resistance for anode current calculation (ohm)

Re = Reynolds Number (dimensionless)

S = number of cycles in stress block (dimensionless)

S_L = factor of safety on lateral stability (dimensionless)

S_t = Strouhal number (dimensionless)

t = nominal wall thickness (mm)

t_d = design wall thickness (mm)

t_{min} = minimum wall thickness (mm)

t_a = ambient sea temperature (min) (°C)

t_p = temperature of pipe contents (max) (°C)

T = mean axial tension (N)

T_{ass} = associated wave period (s)

T_d = Temperature derating factor

T_F = temperature operating factor

T_q = torque applied to pipeline (N.mm)

T_p = peak period (s)

T_z = zero upward crossing period (s)

u = utilisation factor of anode (dimensionless)

U_c = horizontal steady current velocity normal to pipe axis at a distance D_t above seabed (m/s)

U_d = design horizontal water particle velocity normal to pipe axis (m/s)

U_w = wave induced horizontal water particle velocity normal to pipe axis at a height D_t above seabed (m/s)

ν = Poisson's ratio (dimensionless)

V_r = reduced velocity (m/s)

V_k = kinematic viscosity of sea water (m^2/s)

V_m = Maximum orbital velocity due to wave motion (m/s)

W_{AN} = net mass of anode (kg)

W_{eff} = submerged self weight of pipeline and overburden soil per unit length of pipe (N/m)

W_{SB} = submerged unit weight of backfill (N/m)

W_{sub} = submerged weight of pipeline (N/m)

α = coefficient of linear thermal expansion of carbon steel (/°C)

Z_c = Compressibility factor (dimensionless)

Z = steel pipeline section modulus (m^3)

σ_{eq} = equivalent stress (Pa)

σ_h = hoop stress (Pa)

σ_i = installation stress (Pa)

σ_L = longitudinal stress (Pa)

σ_{max} = maximum applied stress (Pa)

σ_y = specified minimum yield stress (Pa)

ΔT = Temperature differential between max. operational and installation (°C)

η = usage factor (dimensionless)

τ = torsional shear stress (Pa)

Φ = resistivity of the electrolyte (ohm.cm)

δ = generalised logarithmic decrement of damping

δV = driving voltage to polarised steel (V)

ρ_{ss} = submerged unit weight of soil per unit volume (N/m^3)

ρw = mass density of sea water (kg/m^3)

μlat = lateral pipeline/seabed friction coefficient (dimensionless)

μ_{long} = longitudinal pipeline/seabed friction coefficient (dimensionless)

μ_{mod} = modified lateral friction coefficient (dimensionless)

TABLE OF CONTENT

CHAPTER 1
INTRODUCTION

1.1 Background of Study

Pipeline is one of the commonly used methods to transport liquid and in Oil and Gas industries pipeline engineering is one of the vital components to be taken into accounts because it involve a huge amount of investment and it's also transport liquid that valuable and also harmful to environment.

The development of pipeline engineering thru out the time is mostly by initiative of the major world's Oil and Gas Company such as Shell from Holland, Det Norsk Veritas (DnV) from Norway and also American Petroleum Institute. These major entities had dominated the development of pipeline engineering and event our PETRONAS Technical Standard or PTS also is based on the SHELL and DnV standard.

These standards are being developed based on the theory and experiences and most of it involve assumption and prediction especially for the submarine pipeline. This is because underwater pipeline is hard to be monitored physically compared to onshore pipeline. Besides that, submarine pipeline also encounter a lot of uncertainties due to fact that it is under the sea which have a different environment and condition to be considered.

As the submarine pipeline is a relatively new industry, there is lack of a set of systematic standard that can be employed to optimize pipeline development projects. Pipeline operating companies are looking more and more to engineering innovation to provide them with cost-effective approaches for developing their pipeline systems. Because of the huge investment in offshore, especially in deepwater development, any experience gained from any pipeline project is very valuable to the whole industry

1.2 Problem Statement

The submarine pipeline is exposed to the hydrodynamic force which consists of the drag and lift force. This force can cause instability to the pipeline and its can either cause the pipeline to be drag, vibrated or event worst the pipeline can event buoy. A moving pipeline is not very favorable because due to the fact that the pipeline is steel and this continuous movement can cause fatigue which result to the pipeline failure.

1.3 Objectives and Scope of Study

The objective of this project is to verify and evaluate both empirical and theoretical procedures in the prediction of On Bottom Stability of the submarine pipeline. This include pipeline at various depth and also shore approaches pipeline under the influence of waves and current.

A real life submarine pipeline project will be identified and all related data regarding its on bottom stability will be collected. Then a model of a submarine pipeline will be developed based on the real life project. The model then will be tested in the Offshore Laboratories under environmental condition as stated in the real life project. The result from the test will be compared with Carigali's On Bottom Stability Spreadsheet and theoretical evaluation.

CHAPTER 2

LITERATURE REVIEW AND THEORY

2.1 Submarine Pipeline

Generally submarine pipeline can be classified as rigid pipeline and also flexible pipe but due to certain criteria such as economy and maintenance rigid pipeline in much more applicable in this region.

For a rigid submarine pipeline, stability is one of the main concerns because a continuous movement can cause fatigue to the structure. Different from the onshore pipeline, submarine pipeline is hard and costly to be repaired if anything happened. Besides that, submarine pipeline also are exposed to a continuous hydrodynamic forces due to the wave and current. For this reason, pipeline need to be design so that it will remain stable at the seabed under various condition from empty pipeline during installation phase up to heavy storm during operation period.

Hydrodynamics stability is determined using Morison's Equation which relates hydraulic lift, drag, and inertial forces to local water particle velocity and acceleration. The coefficients used, however, vary significantly from one situation to another. For example, the lift and drag coefficients of 0.6 and 1.2, which are representative of a steady current, are not appropriate for oscillating flow in a wave field. In addition, these coefficients are reduced if the pipe is not fully exposed because of trenching or embedment.

There are several methods can be implemented for pipeline to achieve its hydrodynamic stability which is:

- Increase the weight of pipeline by adding external concrete coatings.
- Increase the submerge pipeline by increasing the pipe wall thickness.
- Increase the stability by trenching the pipeline into the seabed.

- Increase the stability by trenching the pipeline into the seabed.
- Add weight to the pipeline by bolt on the weight or concrete mattresses

And the most commonly used method is concrete coating due to the economic reason and the others methods is usually applied as a mitigation action to overcome the pipeline instability during operation phase.

Figure 2.1: Concrete Coating

Hydrodynamics stability also needs to be analyzed based on the condition that will be exposed to the pipeline which is during Installation Phase, Hydro Test Phase and Operation Phase. These conditions will determine the type of storm and current that will be applied during the analysis.

For the Installation phase, we will use 1-year storm period and same also for the Hydro Test phase. But, for the Operation phase we will used the 100 years storm

Installation condition will became the most governing criteria because at this time the pipeline is empty and very light. Beside that water depth also will have major effect on the hydrodynamic stability. Pipeline at the shallow water will be more affected by the wave and current due to the theoretical wave profile and wave interaction with seabed.

2.2 Modeling Theory

For my pipeline model, it is been advise by my supervisor to use the Froude number in scaling down the prototype pipeline to ensure its suits our limitations. Besides that, Froude number also is more flexible and comprehensive compare to the Reynolds numbers because its covers more criteria compare to the Reynolds.

For a model scale of 1:100 to 1:200, it is virtually impossible to maintain the Reynolds similitude (Chakrabarthi, 1994).

> "... This is one reason why model tests are not always done at exactly equal Reynolds numbers. Some relaxation of the equivalence requirement is often acceptable when the Reynolds number is high. Using a wind tunnel may have been possible..."
>
> \- (University of Leeds)

Since the Reynolds similitude is not a good choice, a further elaboration on the usage of similitude is needed. Therefore, we use Froude similitude, by allowing variations in Reynolds number (Gao, Gu, & Jeng, 2002). As a start, the calculations from the PL 184 SKO pipeline are derived. Using assumed data and applying Froude Number in model scaling:

For the calculation of model:

Taking scale of 1:10;

Pipe diameter (p=prototype, m=model):

$$d_P = \ell_P = 1.00m$$
$$d_m = \ell_m = 0.10m$$

The equations used are as follows:

Froude Number:

$$Fr = \frac{U_m}{(gD)^{1/2}}$$ **Equation 1**

Froude number is the ratio of inertia force to gravitational force.

For KC number:

$$KC = \frac{U_m T}{D}$$ **Equation 2**

KC number is the Hydrodynamic force on the pipe under wave loading.

For Reynolds number:

$$Re = \frac{U_m D}{\nu}$$ **Equation 3**

Since both Fr and Re cannot be satisfied concurrently on model test, Froude scaling is used mainly and variations are allowed for Re up to 2 magnitudes (Chakrabarthi, 1994).

According to Froude's Law

$$\frac{\lambda_{U_m}}{\lambda_g^{1/2}\lambda_D^{1/2}} = 1$$ **Equation 4**

Since $\lambda_g = 1$

$$\lambda_{U_m} = \lambda_D^{1/2}$$ **Equation 5**

$$\lambda_T = \frac{\lambda_D}{\lambda_{U_m}} = \lambda_D^{1/2}$$ **Equation 6**

Therefore;

$$\lambda_{KC} = \frac{\lambda_{U_m}\lambda_T}{\lambda_D} = 1$$ **Equation 7**

This proves that Froude and KC number can be satisfied concurrently in our model test.

The range for Froude and KC number for the South China Sea are 0-0.5 and 0-20, respectively. If the experimental values fall between these ranges, they can be accepted. The Reynolds number is smaller than the actual value by two orders (Gao, Gu, & Jeng, 2002).

Below are the comparison between Froude and Reynolds modeling number based on the phenomenon:

Phenomenon	Froude	Reynolds
Water plane area	$1: \alpha^2$	$1:\alpha$
Wetted hull surface	$1:\alpha^2$	$1:\alpha^2$
Volume of displacement	$1:\alpha^3$	$1:\alpha^3$
Mass of displacement	$1:\alpha^3$	$1:\alpha^3$
Mass moment of Inertia	$1:\alpha^5$	$1:\alpha^5$
Pressure in the fluid	$1:\alpha^2$	$1:\alpha^{-2}$
Fluid forces	$1:\alpha^3$	1:1
Fluid moments	$1:\alpha^4$	$1:\alpha$
Forward speed	$1:\alpha^{1/2}$	$1:\alpha^{-1}$
Propeller diameter	$1:\alpha$	$1:\alpha$
Propeller pitch (deg)	1:1	1:1
Wave length	$1:\alpha$	$1:\alpha$
Wave period	$1:\alpha^{1/2}$	
Oscillation frequency	$1:\alpha^{-1/2}$	
Vertical displacement	$1:\alpha$	$1:\alpha$
Angular displacement	1:1	1:1
Vertical velocities	$1:\alpha^{1/2}$	$1:\alpha^{-1}$
Angular velocities	$1:\alpha^{-1/2}$	
Rpm	$1:\alpha^{1/2}$	1
Power	$1:\alpha^{4\frac{1}{2}}$	
Vertical acceleration	1:1	1:1
Angular acceleration	$1:\alpha^{-1}$	

CHAPTER 3
METHODOLOGY

3.1 Project Identification

For this project, a real – life subsea pipeline project had been identified and it is PL 184 SKO pipeline replacement project. Petronas Carigali Sdn. Bhd. (PCSB) has planned to replace the old pipeline system facilities within Temana Field. This offshore field is located in Balingian geological province, some 30 km west offshore Bintulu, Sarawak. Temana field comprises two main production platforms, TEP-A (Sector A) and TEP-B (Sector B).

Temana-A consists of production platform TEP-A bridge-linked to drilling platform TEDP-A and compression platform TEK-A. In this sector are:-a remote vent platform TEV-A and 3 satellite platforms TEJT-CC, TEDP-E and TEJT-T all connected via pipelines to the TEP-A producing platform. Temana-B consists of production platform TEP-B bridge-linked to drilling platform TEDP-B. In this section are: - remote vent platform TEV-B, and two satellite platforms TEJT-C and TEJT-W, all connected via pipelines to the TEP-B producing platform.

The crude oil from TEP-A is pumped to the TEP-B platform by three (3) crude oil transfer pumps P-803 (7543bpd) and P-804/805 (2x16011 bpd) ,the combined crude oil is exported to Bintulu Crude Oil Terminal (BCOT) by the 2 x 16,000 stb/d Crude Oil Transfer Pumps (COTP).

The old pipeline system facility to be replaced is:

- 12-inch Crude oil Pipeline from TEP-B to BCOT (PL184)

The pipeline replacement work is performed by PCSB's Development Division (DD) themselves. The personnel involved are from Facilities Engineering Department (DFE), Supply Chain Management Department (DSCM) and Corporate Health, Safety and Environment Department (DHSE) under PCSB.

(DFE), Supply Chain Management Department (DSCM) and Corporate Health, Safety and Environment Department (DHSE) under PCSB.

The PCSB's Sarawak Operations (PCSB-SKO) will be assisting the project team for the entire work.

The work consists the scope of:

- Detailed engineering design
- Procurement
- Construction
- Commissioning

The crude oil pipeline from TEP-B to BCOT including the pig traps shall be designed in accordance with PTS 20.196 and its supplementary issued in February, 2004, which specifies the ANSI/ASME B31 code in order to comply with the requirement of the Petroleum (Safety Measures) Act, 1984.

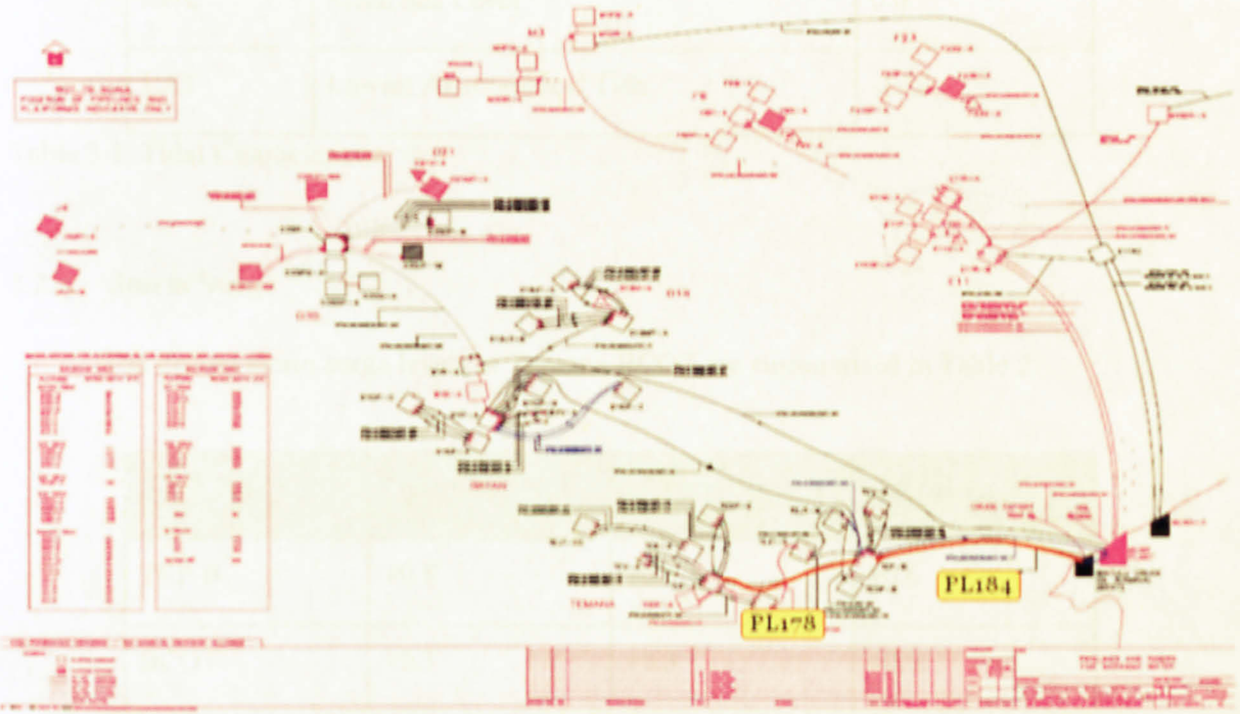


Figure 3.1: Location of PL 184 in Temana Field

3.2 Data Gathering

And following is this PL 184 stability related data:

3.2.1 Water Depth

Relative to MSL the water depths in the vicinity of the platforms are as follows: -

TEP-B Platform = 28.82 m

Shore Approach = 9.00 m

3.2.2 Tidal Data

The characteristic tidal levels at Temana-BCOT are summarised in table below:

Level	Description	Height(m)
HAT	Highest Astronomical Tide	+0.98
MSL	Mean Sea Level	0.0
LAT	Lowest Astronomical Tide	-1.22

Table 3.1: Tidal Characteristic

3.2.3 Storm Surge

The design storm surge levels at Temana-BCOT are summarised in Table 2:

Platform	1-year	10-year	100-year
TEP-B	+0.3	+0.3	+0.6
BCOT	+0.3	+0.3	+0.6

Table 3.2: Storm Surge

3.2.4 Wave Data

The design 1-year and 100-year omni-directional and directional wave conditions for the platforms are summarized in Tables 3

H_s and T_p are required for Stability Analysis while H_{max} and T_{ass} are required for Spanning Analysis and Riser Stress Analysis

Description	Unit	Return Period	
		1-year	100-year
Significant Wave Height, H_s	m	3.2	4.4
Mean Zero Crossing Period, T_z	s	9.2	10.5
Maximum Wave Height, H_{max}	m	5.8	8.4
Max Period, T_{maz}	s	11.1	13.8

Table 3.3: Omni-Directional Wave Data

Note:

1. *Tp-over-Tz ratio is 1.3*

3.2.5 Current Data

The design 1-year and 100-year design omni-directional current data for the platforms are summarized in Table 4.

Description	Unit	1-year	100-year
Surface(Note1)	m/s	0.55	0.69
Near sea bed(Note2)	m/s	0.37	0.47

Table 3.4: Omni-Directional Steady State Current Profile at TEP-B

Notes:

1. "Surface "currents were values at 5m below MSL
2. "Bottom" currents were values at 2.5m above seabed

3.3 Hazard Identification

Hazard identification is necessary to avoid implications later on when doing the experiments. Identifying hazards before it occurs often can save time, money and even life. There are a few vital areas that had been identified as hazardous, and a few steps had been taken as a cautionary measure. They are:

3.3.1 Noise

These experiments will require the usage of a powerful pump that generates a lot of noise. To counter the side effect of noise, ear mufflers will be used, and the pump had been isolated during the installation of the flume.

3.3.2 Vibration

There will be a lot of vibration by the pumps that generate currents for our flume. Therefore precautionary steps have been taken by padding the pump area (done during pump installation).

3.3.3 Electrical

As the experiments will mainly use high electricity power to operate the pump, some cautionary steps have been taken:

1. Isolate the plug from water tank/pump.
2. Use rubber insulator to cover the switchbox in case of overflowing of water tank.
3. Only operate the pump when proven necessary.
4. Use rubber boot when entering the tank and if necessary avoid contact with water.

3.3.4 Dust

No dust hazard identified in the lab experiments.

3.3.5 Fire and Explosion

Although most of the equipments use water, fire and explosions hazard do exist as the pump uses high electricity energy. Since the nature of fire hazards in my experiments are water-electricity related, conventional water-based fire extinguishing plan is unsuitable. Instead, dry-chemical and foam-based fire extinguisher are prepared as a contingency plan.

3.3.6 Falling Object

This may occur during lifting process, if the equipment is not being properly checked and maintain. It is a best practice to avoid from being under the lifted object to avoid any unwanted thing happen

3.3.7 Tilting

The space at the side of the tank is very small and sometimes people need to step on it to reach some area. Beside of small area to be step on at the middle of it have a steel structure elevated which can cause tilting hazard.

3.4 Experimental Setup

3.4.1 Modeling and Layout

The model will be fitted onto the wave tank bed. The pipeline models are PVC pipes, 1.5m long each. Three different diameters are used; 6", 4" and 3" after we decide to scale it down by using Froude modeling number of 3. The end of pipeline will be cap to simulate installation condition which is the most governing condition in pipeline on bottom stability.

Sand and stones are used to fill up the pipeline model to simulate self weight of the pipeline. If the sand and stone are not sufficient we may change it to the steel pipe or mud.

Two 0.01 sensitivity digital scales are used to measure the force acting on the pipeline, the model and the digital scale will be connected using a tensioned cable.

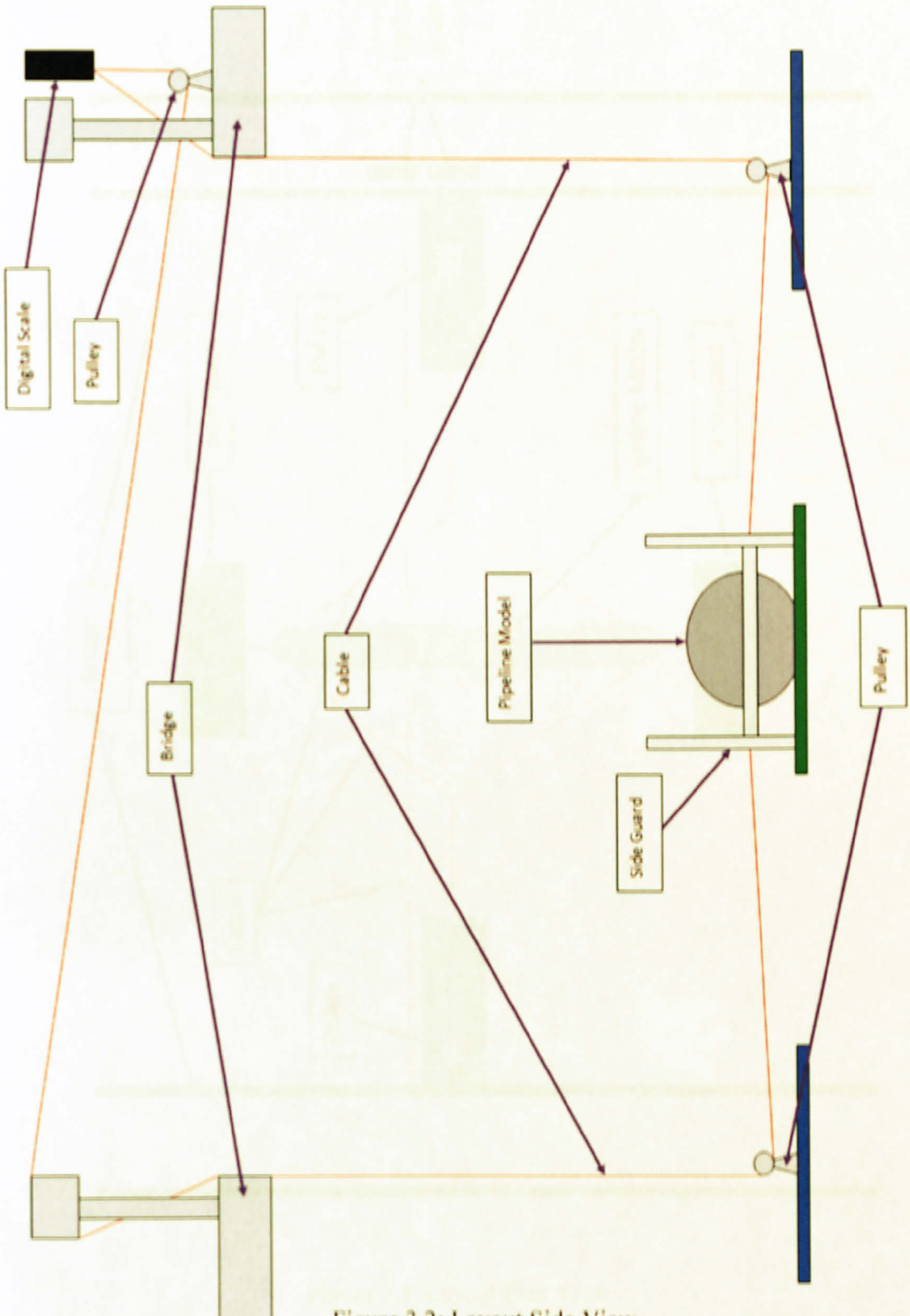


Figure 3.2: Layout Side View

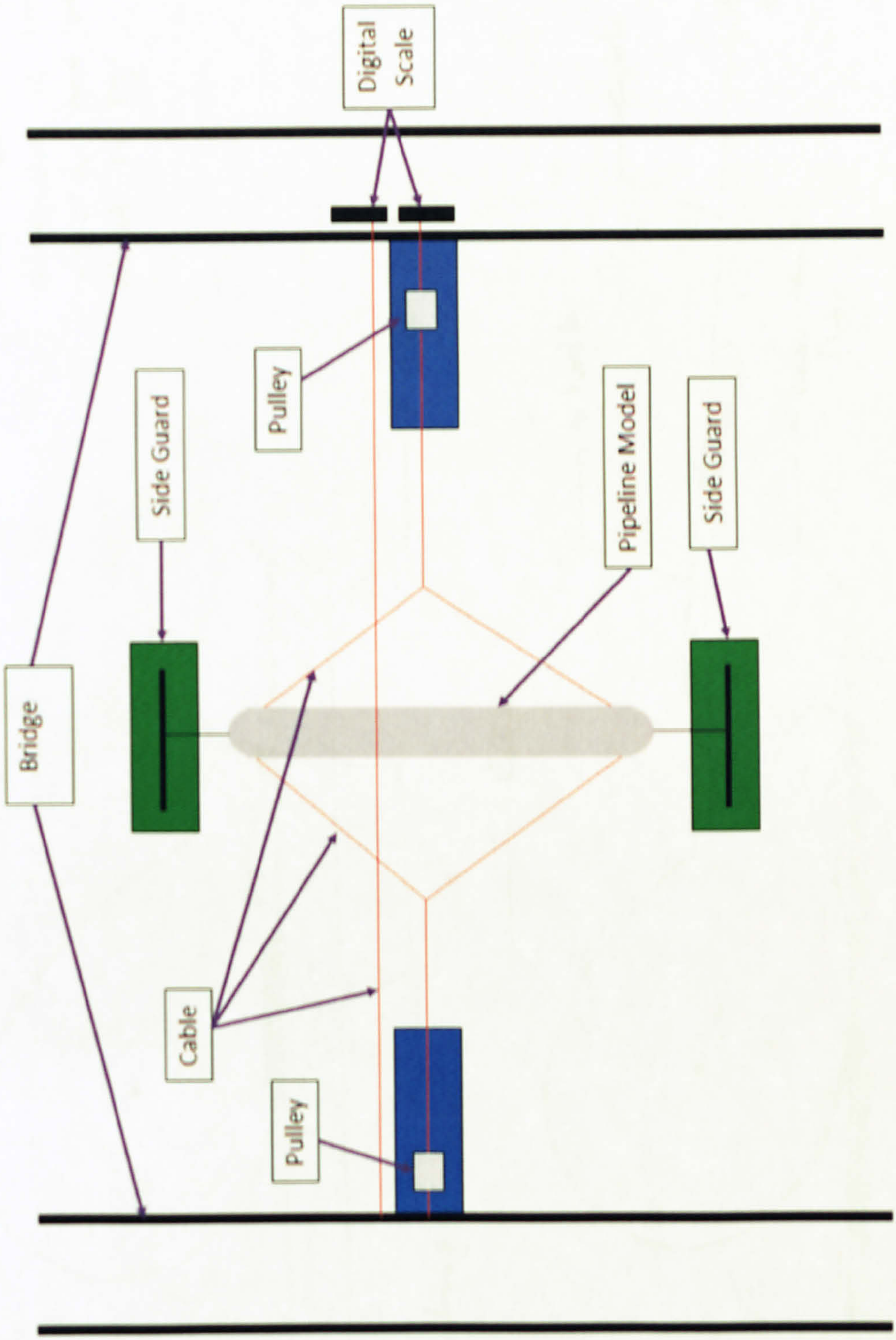


Figure 3.3: Layout Plan View

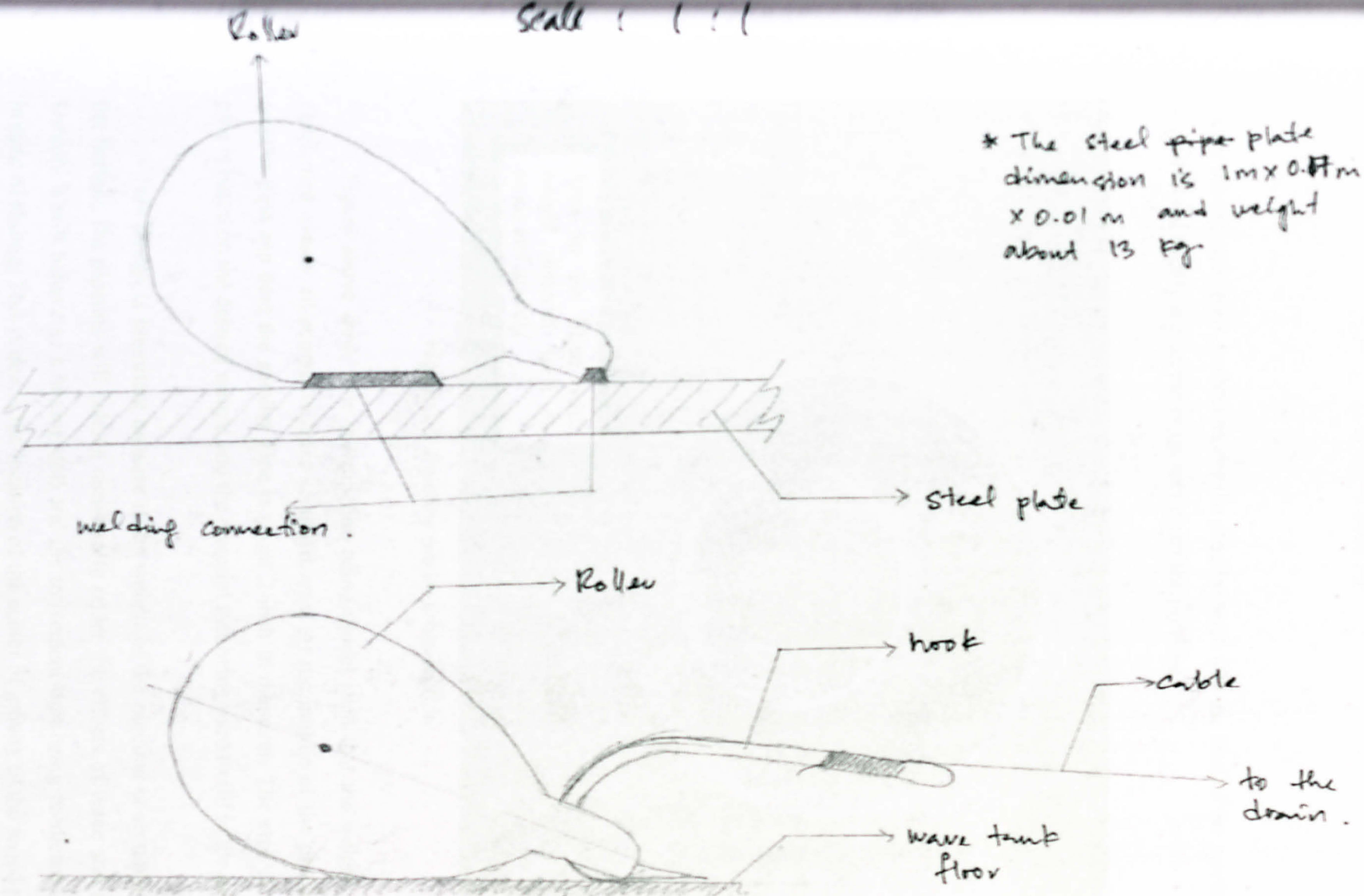


Figure 3.4: Roller Design

As for the pipeline itself, we had model the pipeline such that the load of the pipeline will be distributed evenly across the cross section of the pipeline.

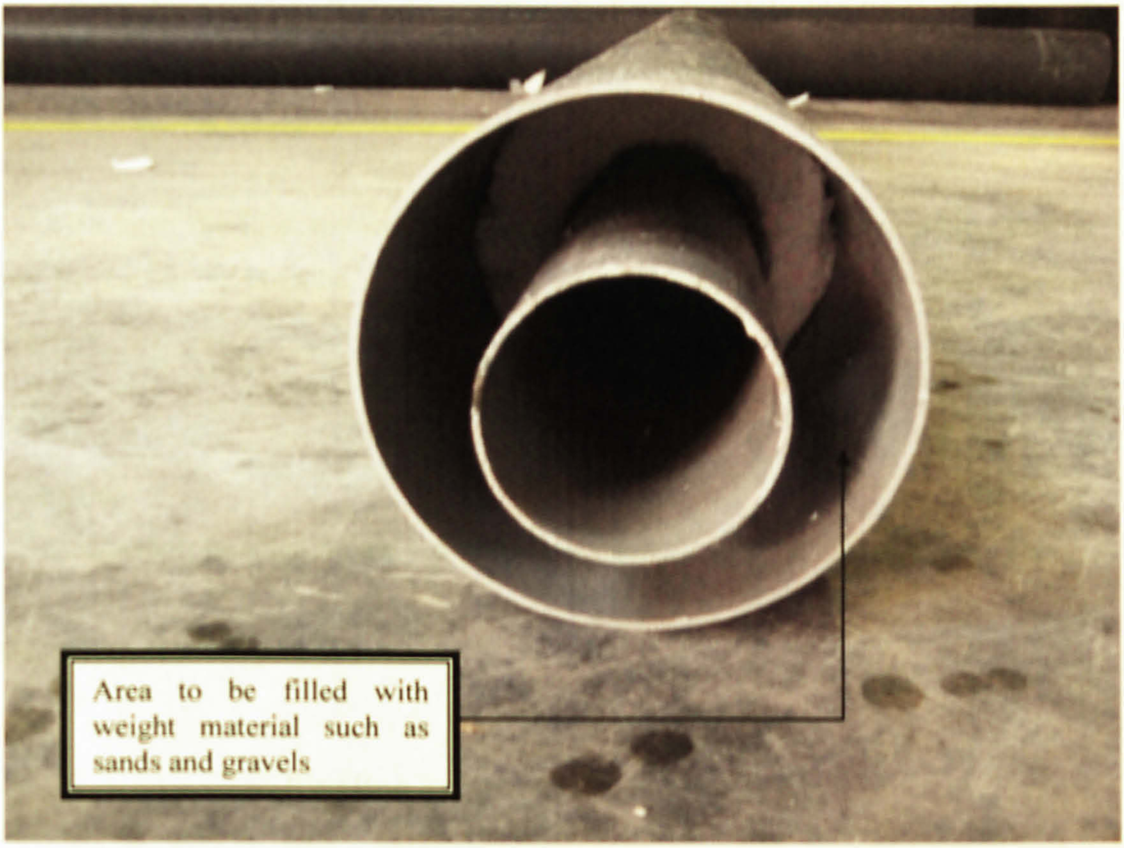


Figure 3.5: Pipeline model cross section

Figure above show how the pipeline being model such that the weight will be distributed evenly all across its cross sectional area. At the middle of the pipeline is a smaller pipe and here the smaller pipe is about 2 inch in diameter. The size of smaller pipe is based on the desired weight and the material that being used to fill in the pipeline.

This design is important because if the weight of the pipeline is accumulating at the bottom, the pipeline will behave unnaturally under the effects of wave and current. Usually it will behave as a boxing doll and the movement that being predicted is wake instead of sliding. This is due to the location of the center of gravity of the tested pipeline model. To simulate a real pipeline the center of gravity of the model should be near the middle of the cross sectional area.

For submarine pipeline purpose the pipeline should be model so that it will submerged under the water. This can be achieve by having the pipeline weight at least same with its buoyancy.

To have the idea with this minimum required weight some calculation can be done by having assumption of minimum required weight is equal to the pipeline buoyancy.

Assumption:

Minimum required weight = pipeline buoyancy

Then the pipeline buoyancy is:

$$Pipeline\ Bouyancy = \frac{\pi D^2}{4} \rho_{water} L$$

$$\rho_{water} = 1000 \frac{kg}{m^3}\ as\ the\ test\ is\ in\ the\ wave\ tank\ with\ fresh\ water$$

$$D = Pipeline\ Diameter = 4.33\ inch\ (\ by\ measured) = 11\ cm = 0.11\ m$$

$$L = Pipeline\ Length = 1.5\ meter$$

$$Pipeline\ Bouyancy = \frac{\pi (0.11)^2}{4} (1000)(1.5)$$

$$Pipeline\ Bouyancy = 14.25\ kg$$

Because of the cap also provide some buoyancy and it is hard to be measured its volume then it is necessary to just increase the weight of the pipeline, here the weight of the pipeline model that being applied is 18 kg.

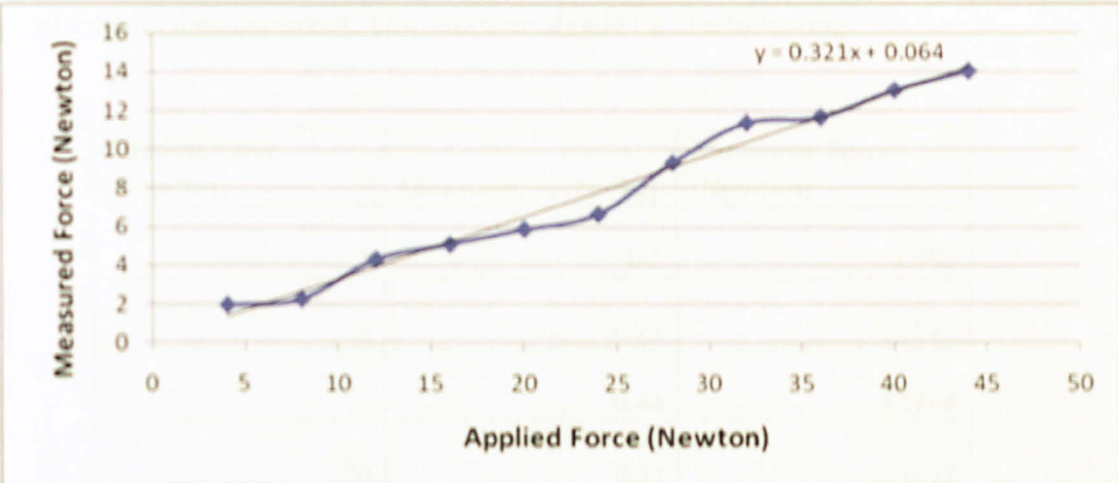
3.4.2 Force Measurement Mechanism

The modeled pipeline will be tested in the wave tank under the wave and current effects. Therefore a mechanism had been design to measure the force that acting on the tested pipeline.

The mechanism is design such as both direct and backward force can be measured. The acting force is measured using a digital scale in unit of kg over 1.5 m length.

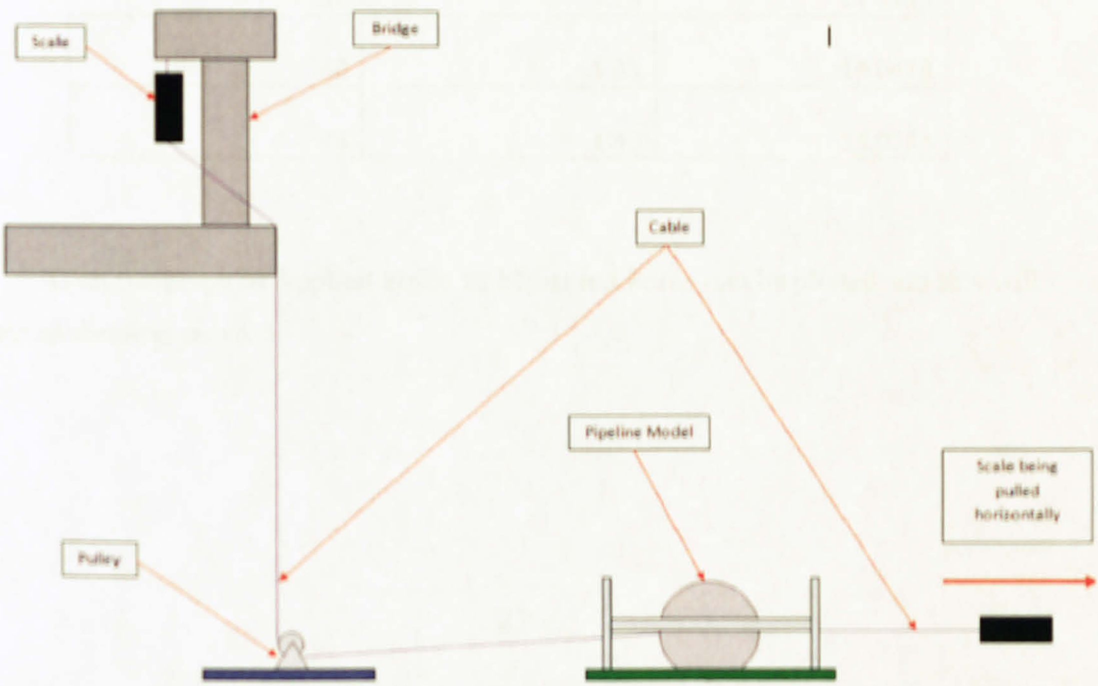
Figure 3.6: Digital Scale

The scale that being used here had been calibrated so that any friction that occurs in the mechanism is taken into account hence we can conclude that the value that being record is an actual force cause by the design wave and currents.



Graph 3.1: Calibration Curve

This is the example of the calibration curve that obtain. To calibrate the scale, we need to do it during the dry condition of the wave tank. How it is done is showed in picture below.



For calibration purpose the pipeline is installed as figure above and one of the scale will be pulled horizontally for certain value of force for example 2 kg, 4 kg and 6 kg and this will be considered as applied force to the pipeline.

Any reading that measured at the scale attached to the bridge corresponding to the applied force is then measured. The data then should be like following:

Applied Force (Newton)	Measured Force (kg)	Measured Force (Newton)
4	0.2	1.962
8	0.23	2.2563
12	0.44	4.3164
16	0.52	5.1012
20	0.6	5.886
24	0.68	6.6708
28	0.95	9.3195
32	1.16	11.3796
36	1.19	11.6739
40	1.33	13.0473
44	1.43	14.0283

Then the graph of Applied Force vs Measured Force can be plotted and this will be our calibration curve.

The purpose of using digital scale is that it is much easier to record the scale reading for further analysis compare to the spring scale. This will reduce the error during the analysis of the testing result.

Beside that I also incorporate with side guard to my mechanism. This main purpose of side guard is to ensure the both end of the pipeline model will move parallel to each others.

This matter is important to ensure the force acting on the pipeline is acting on the desired angle during the test. If the angle change, then the reading also change and if the guard is not used we may have error in our data recording.

Figure 3.8: Pipeline attach to side guard

The side guard is designed so flexible so that it will allow horizontal movement of the pipeline and also vertical movement if the pipeline is about to move is such direction.

This can be done because we a using a roller at the side guard to allow horizontal movement and the pipeline is attach to the side guard thru a cable that allow any vertical movement of the pipeline as the connectors is flexible.

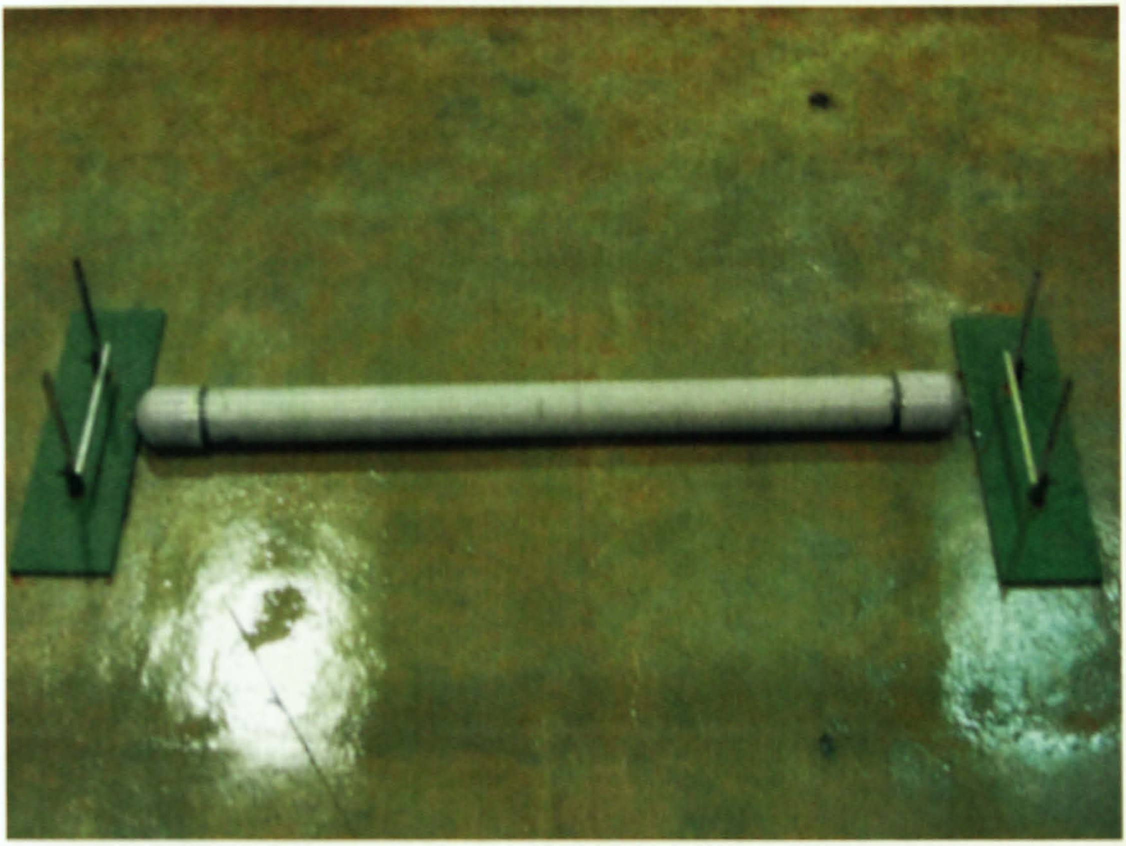
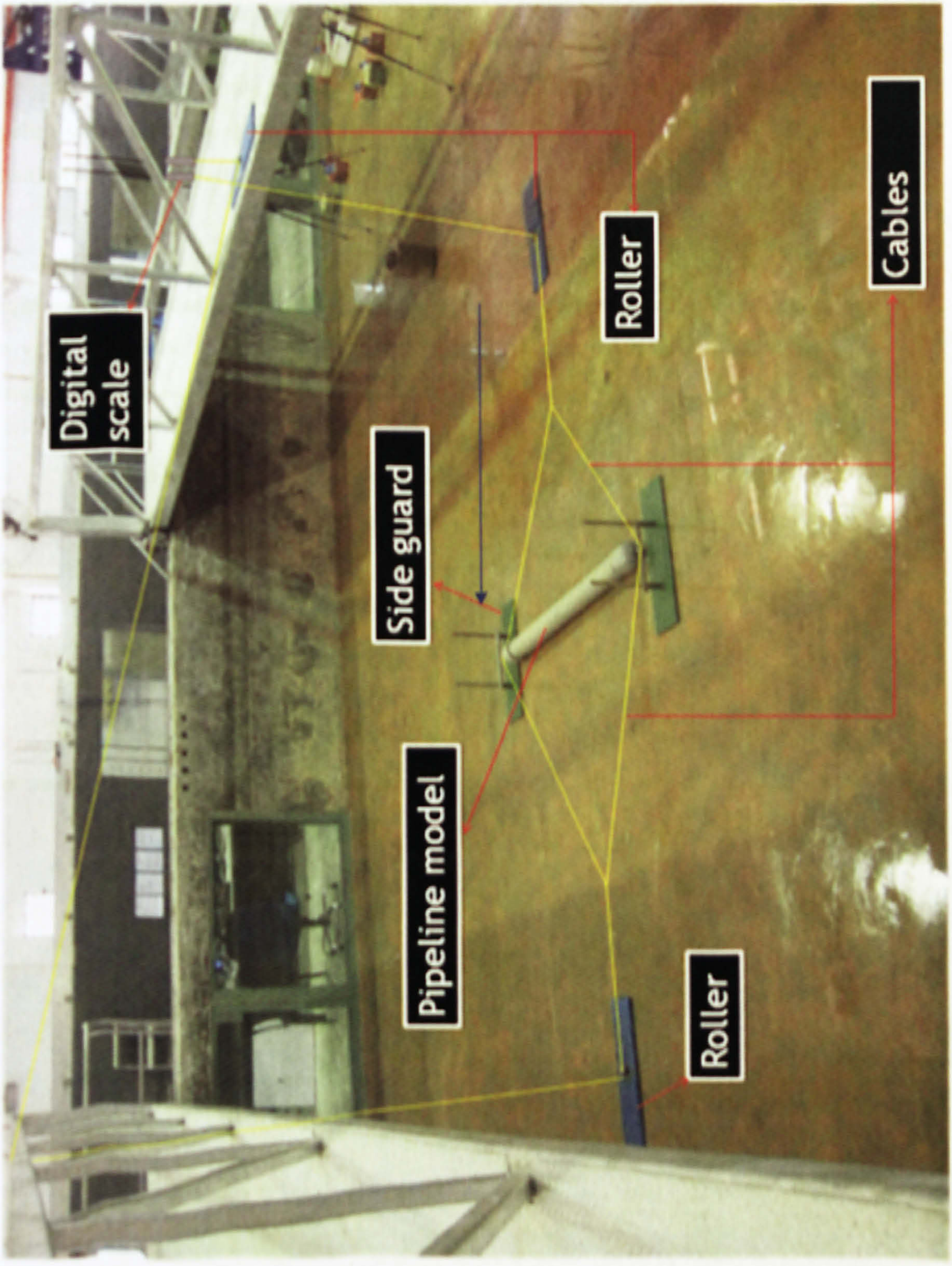


Figure 3.9: Side guard application

Beside that the height of the guard can be adjusted and it is design such away so that in future if we want to corporate with different sedimentation as a bed profile then we can adjust it base on the sedimentation thickness.



The figure shows the full layout of the force mechanism in the wave tank. The cable that attaches the pipeline model with the digital scale should be always on the tension. Here what should happen is under the normal condition which is no wave and current the digital scale should have a small and fix reading such as 0.1 kg.

Because of the cable is on the tension, any small movement cause by wave to the pipeline will be read by the scale and here we have a little limitation as the digital scale sensitivity is only up to 0.01 kg.

At the end of the test, any value recorded from the scale should be minus the initial value that cause by the cable tension. As the raw data recorded in video mode then the value should be taken in forces over a period of time such as forces in each one minutes. Then we will average the value for a final answer.

The result from the experiment actually represents the net force acting on the pipeline under the tested wave and currents. This can be expressed by the equation below:

$$Net\ Force = Drag + Inertia - \mu(Pipeline\ weight - (Bouyancy + Lift))$$

Where μ the friction coefficient between the pipeline and the contact surface is in this case is the concrete floor

3.4.3 Experimental Procedure

To run the experiment, there are certain steps to be follow to obtain a reliable data. For the experiment to run smoothly, it required at least two persons to run the experiment and walkie talkie as a communication medium.

The steps are as following:

1) The pipeline model is installed as shown in the previous figure.
2) Then the scale is turned on and the cable that connected to the pipeline is attached to it. Make sure that the scale show some reading as this will indicate the cables are in tension.
3) Switch on the camera and start recording with initial part by said something or display on board any of the test parameter so that it will be easy to extract the data afterwards.
4) The camera then need to be keeps on and running, beside that it is important to shut down the laboratory door to avoid glare at the scale which will make it hard to be read from the clip afterward.
5) Ask the second person in the control room to start the wave and for current the person that looks after the camera can just go and switch on the pump as it is just at the left side of the wave tank.
6) Record the experiment for about four minutes before stop the recording and after the recording it is a best practice to switch off the camera to preserve the batteries for further experiment.
7) Then go to the side of the wave tank at the screen and run the experiment with the same parameters with previously and record the behavior of the pipeline from the screen. This will give side view of the pipeline movement and further we can justify the types of movement rather it is sliding or rolling.
8) For the side view of the movement also record for about 4 minutes. Don't forget also to record the wave height with some scale at the screen.

9) After finish recording, transfer all recording into computer and cut the recording into smaller clip base on the parameter of the test. This can be done using Ulead software.

10) Play each clip using window media player classic or other media player and run it base on frame. Here we can see the reading for each second.

11) The data that being extracted then need to be times with the calibration factor so that the value that obtain is the value of force of acting on the pipeline regardless of the friction that occur in the mechanism.

12) From the final data, we can plot the force versus time graph.

3.4.4 Lab Setup

3.4.3.1 Wave Setup

Below are the steps to setup a wave at UTP wave tank:

1) Open Wave Maker at the computer in the control room
2) Click "Setup" at the toolbars
3) Click "Project"
4) Insert desired water depth
5) Model Scale default 1
6) Click "Sea State" at the tool bars and choose desired condition
7) Click "New"
8) Insert wave parameter such as height and frequency
9) Description is optional
10) Click "ok" and "yes"
11) At the "Output" choose "Dry Run Std PTF's"
12) Click "Run"
13) When the wave achieve max semi struts click "stop"
14) At the "Output" choose "Paddles Std PTF's"
15) Click "Run"

3.4.3.2 Current Setup

Below are the steps to setup current at UTP wave tank:

1) Switch on the Laptops
2) Open Ventrino +
3) Click "Serial Port" at the toolbars and choose ventrino
4) Click "Load from Instrument"
5) Open Polysyncs
6) Go to the control panel at the wall
7) Swith on the power
8) Select "B" and "RY" at the analog switch
9) Switch on pumps
10) Adjust frequency using analog controller. For the linear current all the value need to be same
11) Rate is 20 Hz = = 10 cm/s

    25 Hz == 16 cm/s

CHAPTER 4

RESULT AND DISCUSSION

4.1 Result and Findings

Seven tests had been done with 18 kg of 4 inch pipeline model. For the test, the water depth is fixed at 0.5 meter deep but other parameters such as wave height, wave period and current velocity are varies.

The parameters for the test are as follow:

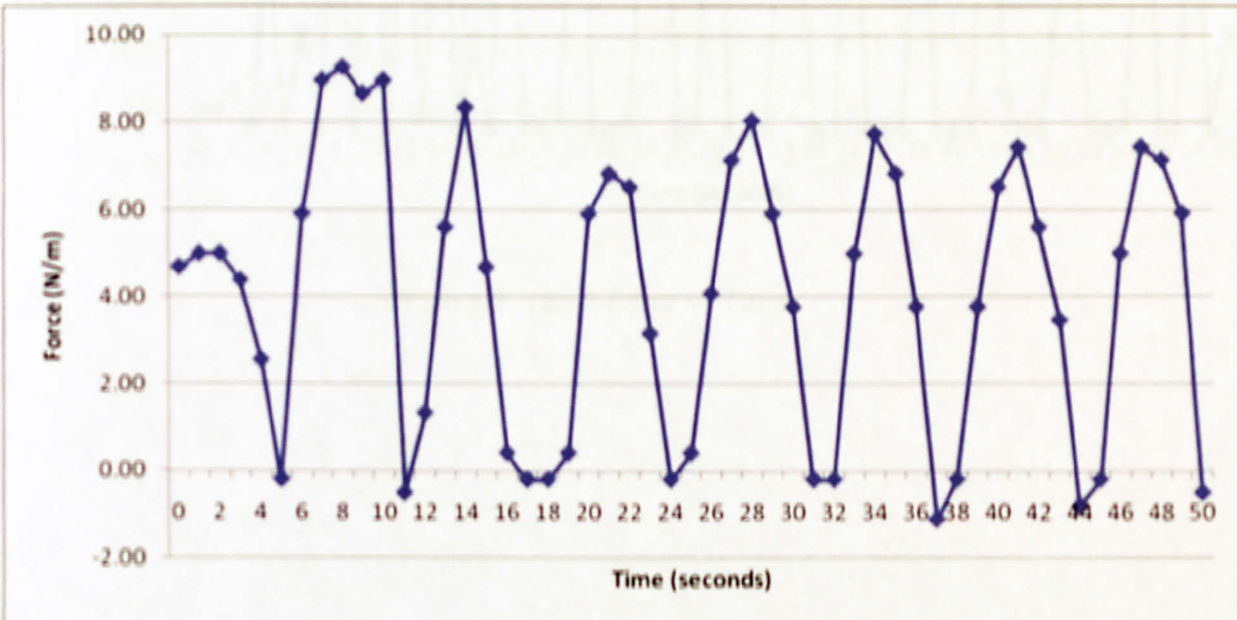
Test	Current Velocity (m/s)	Wave Height (meter)	Wave Frequency (Hz)
1	0.15	0	0
2	0.25	0	0
3	0	0.08	0.15
4	0	0.03	0.15
5	0	0.10	0.5
6	0.2	0.05	0.5
7	0.2	0.08	0.15

And the for the test result, it is found that for test 1, 2 and 4, there are no reading being measured this is probably because the force acting on the pipeline model are to small and this also indicate that under these test parameters the pipeline model are stable at the wave tank floor.

But for other test the result are as following:

Test 3

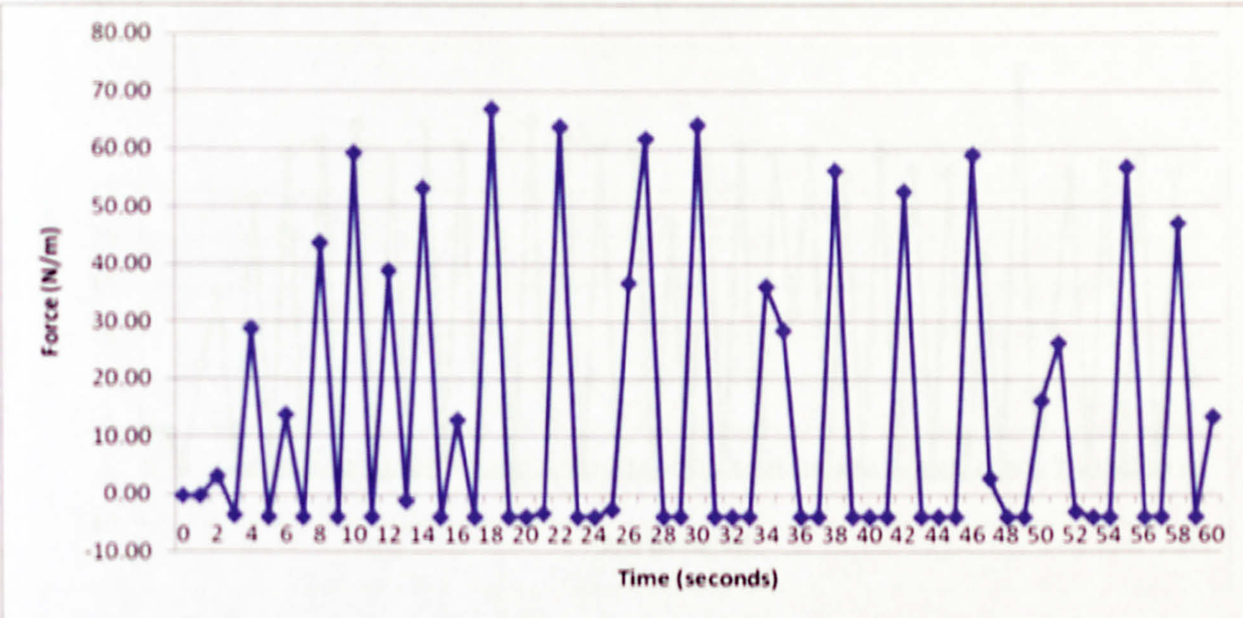
Maximum force that being measured is 6.18 N/m



Graph 4.1: Test 3 Force vs Time

Test 5

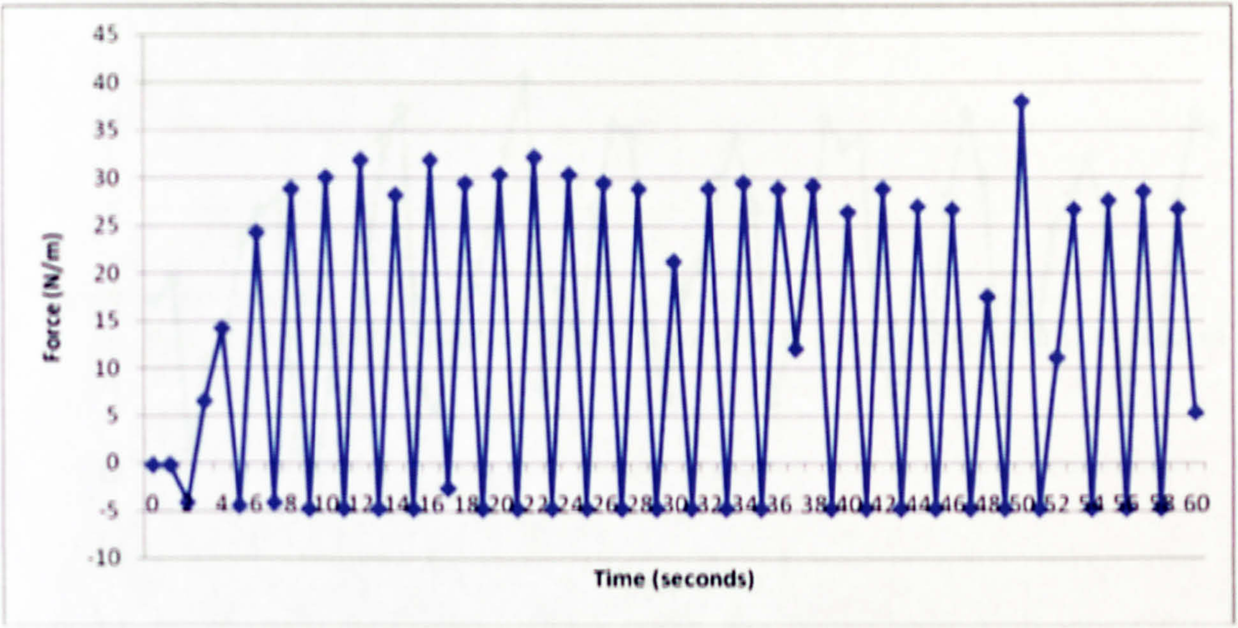
Maximum force that being measured is 44.69 N/m



Graph 4.2: Test 5 Force vs Time

Test 6

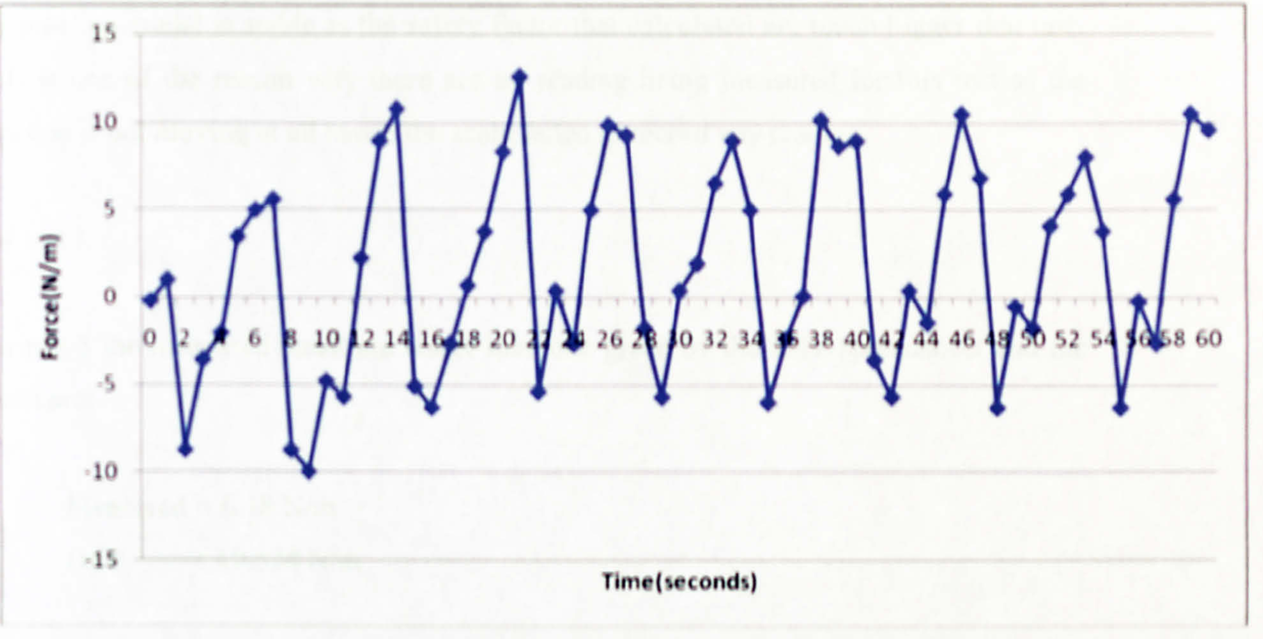
Maximum force that being measured is 25.33 N/m



Graph 4.3: Test 6 Force vs Time

Test 7

Maximum force that being measured is 8.424 N/m



Graph 4.4: Test 7 Force vs Time

4.2 Discussion for Experiment

For the results obtain we further compare it with Carigali (DnV) spreadsheet result and the findings are like following:

For the test 1, 2 and 4 the result are according to the DnV spreadsheet which are the pipeline model is stable as the safety factor that calculated are much bigger that one. This is one of the reason why there are no reading being measured for this test as the pipeline is not moving at all hence the scale failed to record any reading.

For test 3:

For test 3 the measured result are lesser than one given by the DnV spreadsheet and the result are:

Measured = 6.18 N/m

DnV = 19.524 N/m

For test 5:

For test 5 the measured result are bigger than one given by the DnV spreadsheet and the result are:

Measured = 44.69 N/m

DnV = 28.489 N/m

For test 6:

For test 6 the measured result are bigger than one given by the DnV spreadsheet and the result are:

Measured = 25.33 N/m

DnV = 16.359 N/m

For test 7:

For test 7 the measured result are smaller than one given by the DnV spreadsheet and the result are:

Measured = 14.943 N/m

DnV = 25.70 N/m

For test 3 and 7 the result are different because when we are trying to simulate the real condition there are some parameters that we cannot apply such as the real on. For example the friction coefficient that involves in the test and at the real life is very different as the lab are having a concrete bed which are more smooth compare at the real life sea bed condition.

But for test 5 and 6 the result are much bigger because from my observation 0.5 Hz frequency are to severe to me simulate in the offshore lab with water depth of 0.5 m. This is because the pipeline is moving too much and the measured force actually involve the pipeline momentum force as its accelerate quite fast. Therefore for future experiment, the movement of the pipeline need to be record and its acceleration need to be determine so that the force cause by its momentum can be separate to obtain pure force that cause by the environment.

Beside that wave height and current velocity also is a factor that needs to be considering as the laboratory only can support wave up to 1 meter height and current at 0.3 m/s only. Although all of this can be scaled down using Froude modeling theory buy so far this modeling theory didn't represent the real scaled down factor as we had been test this theory for scaled the size of pipeline and also its required submerged weight.

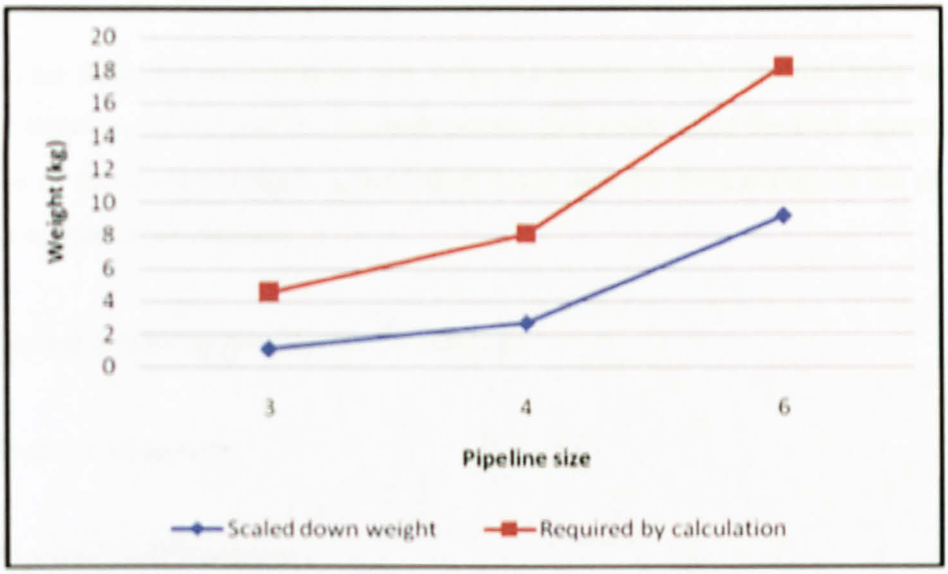
For example if 12 inch pipeline needs at least 72.97 kg of weight for it to be submerged and if this pipeline is model using Froude number and if it is scaled down using a factor of 2, then suppose the 6 inch model pipeline required 9.12 kg of weight to be submerged is following the Froude modeling law the mass will be by factor of $1/\alpha^3$.

But what happen is the pipeline model will be floating and seems the self weight is not sufficient. And when it's come to the calculation, the required submerged weight for 6 inch pipeline is 18.24 kg, nearly double the value from modeling theory.

And if it can be simplified to see the correlation between size and required submerged weight, it can be represent by the graph and table below using 12 inch pipeline as a base case and then scale it down using the Froude modeling theory by a factor of 2, 3, and 4. Here we analyst by the unit length of 1 meter.

Factor	Pipeline size	Scaled down weight (kg)	Required Weight (kg)
4	3	1.14	4.56
3	4	2.7	8.107
2	6	9.12	18.24

Table 5: Comparison between scaled down weight and calculated required weight



Graph 4.5: Comparison between scaled down weight and calculated weight

From the result, we can clearly see that the Froude modeling theory is not representing actual value although by equation, suppose it is. But, the factor that they consider in the Froude modeling theory seems need to be revised back because not all the factor need to be scaled down such us gravitational acceleration and such as because it is a same force acting on the model regardless of its size.

4.3 Discussion for Parametric Study

Apart from the experimental test, some parametric study also had been done to study the behavior of the pipeline towards certain parameter using the DnV spreadsheet. The scope of the study is regarding the safety factor and the force acting on the pipeline as the parameter value changed.

The parameter that being consider for the study is:

1) Pipeline Diameter

2) Pipeline Wall Thickness

3) Content Specific Gravity

4) Concrete Coating Density

5) Significant Wave Height

6) Wave Period

7) Current Velocity

8) Wave Angle with respect to the pipeline

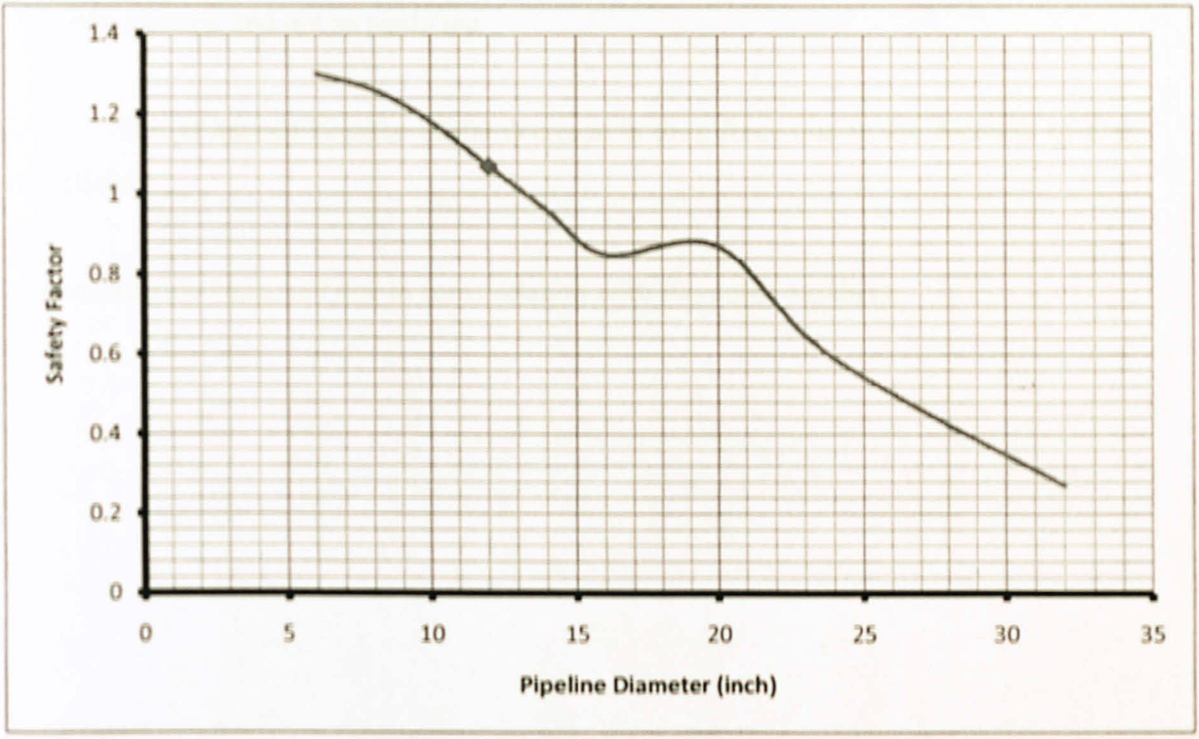
9) Mean Grain Size

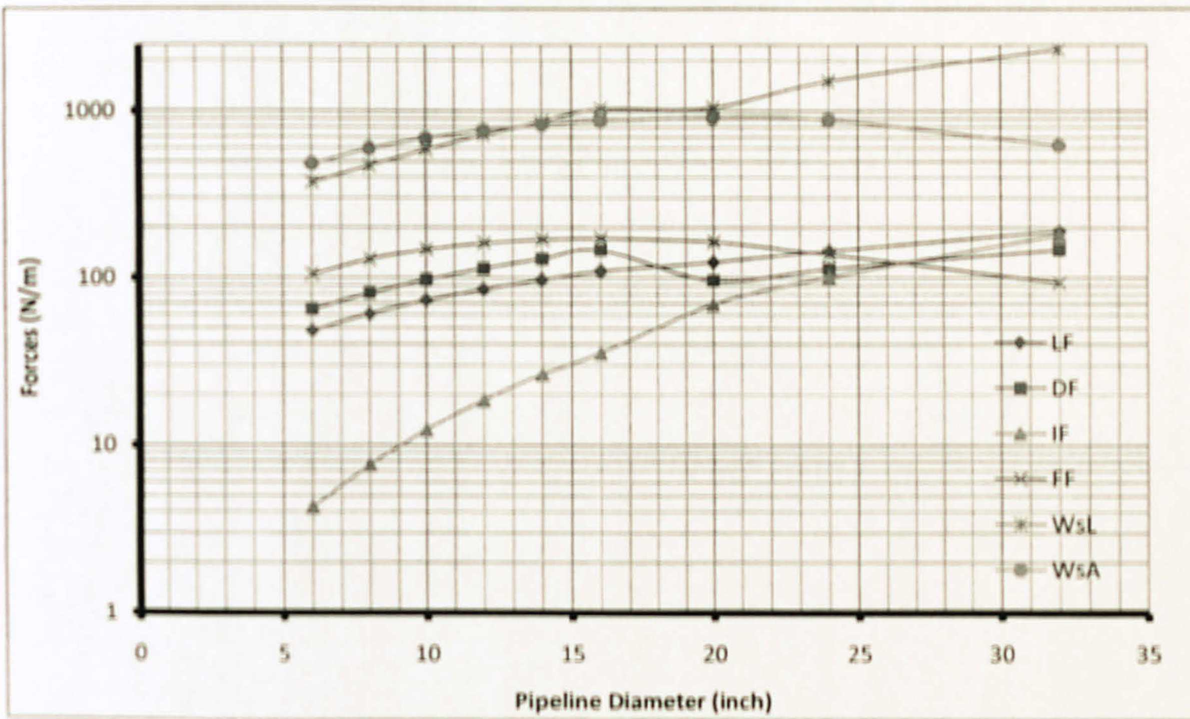
10) Soil Shear Strength

11) Water Depth

The results are as follow:

For Pipeline Diameter;



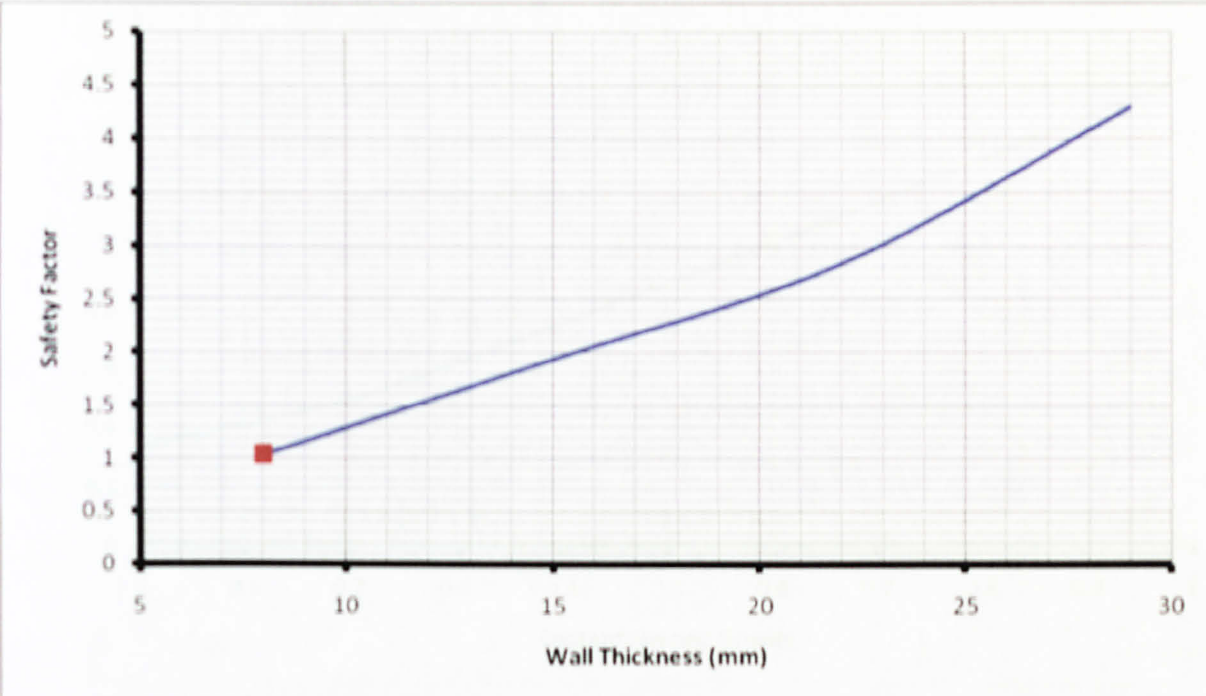


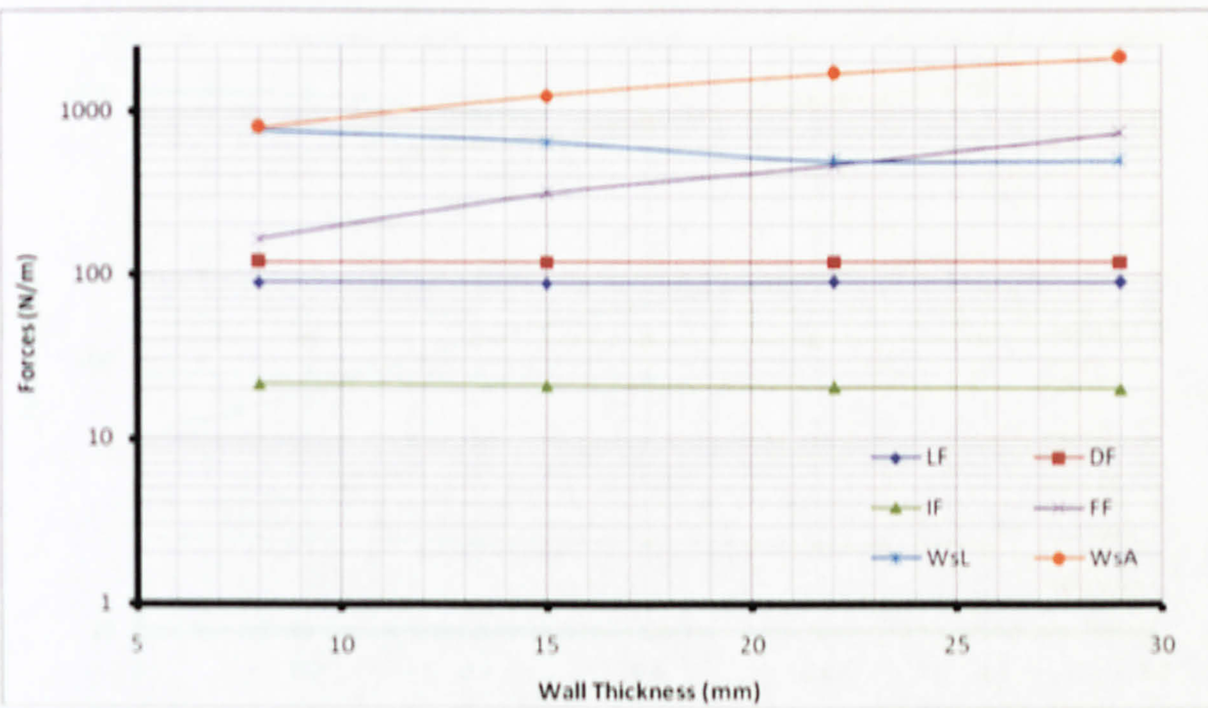
From the analysis we found that the size of 17, 18, and 19 inch, the safety factor slightly increase and not as predicted

This is happen because of the decrease in drag force due to the decrease of drag coefficient

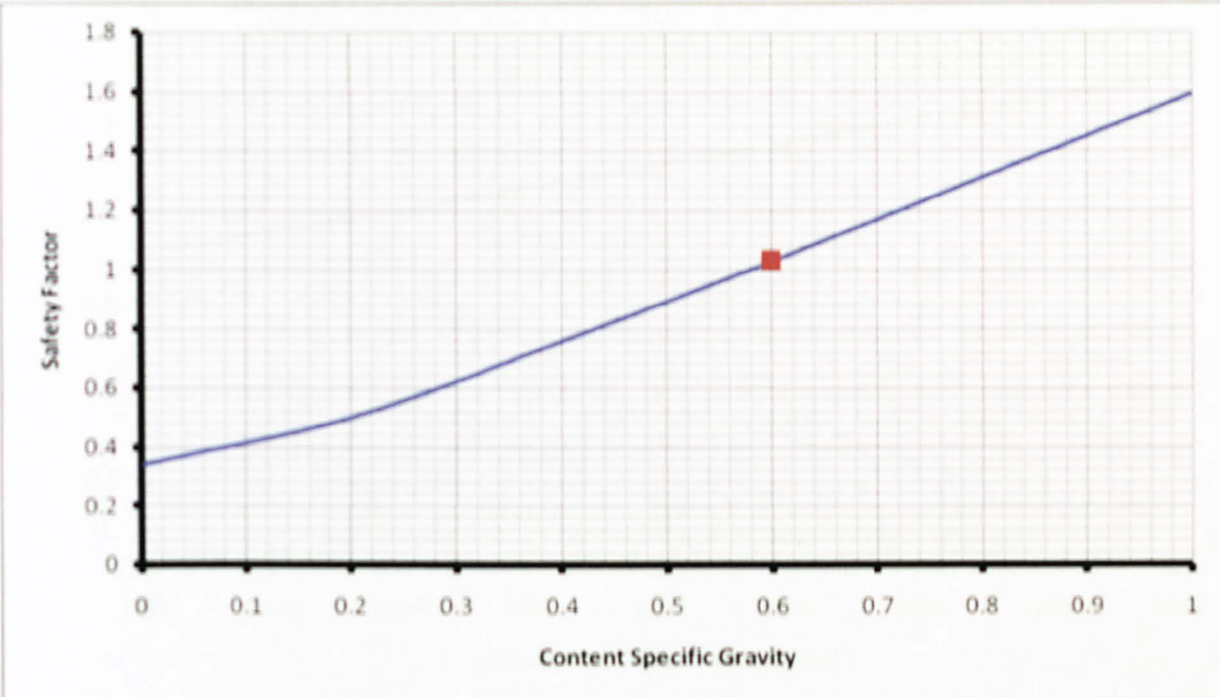
This condition happened due to some relation with Reynolds numbers.

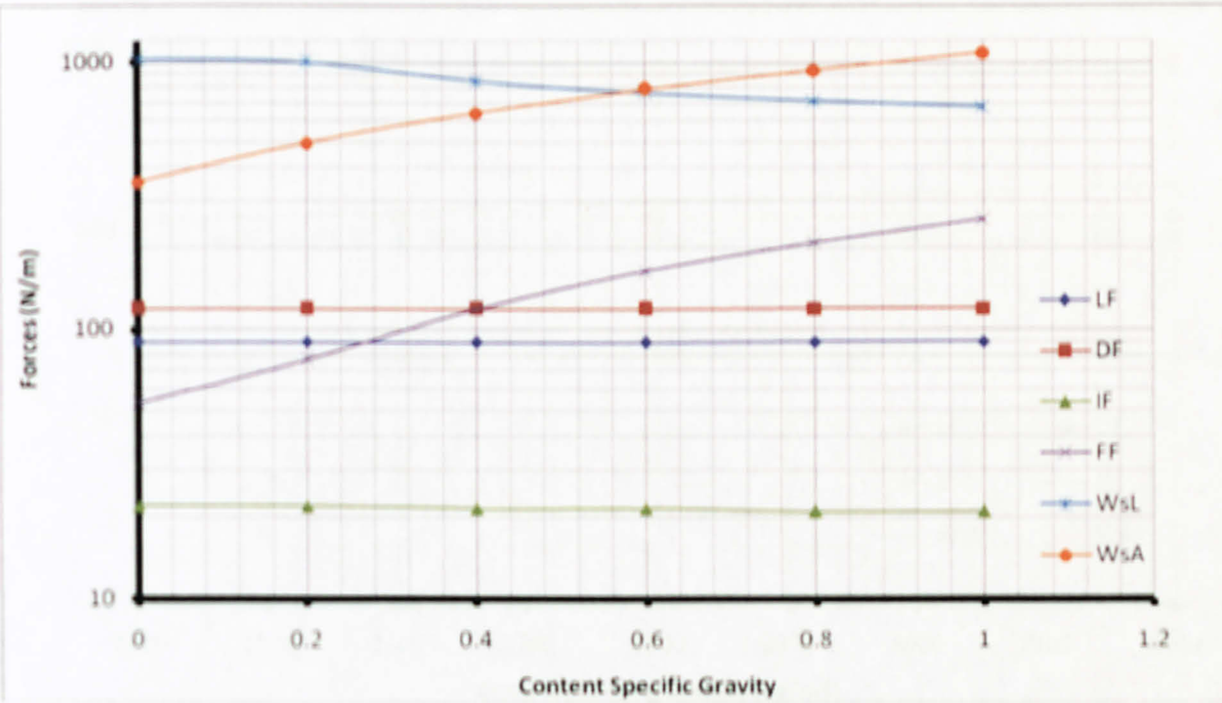
For Pipeline Wall Thickness;



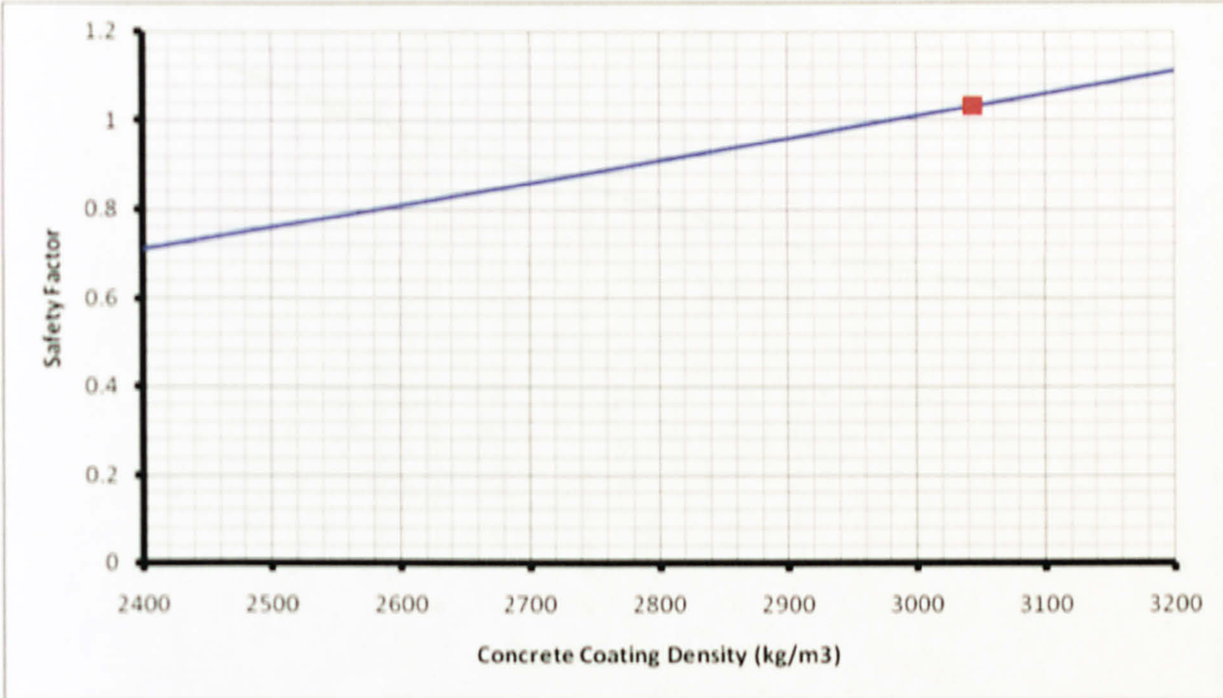


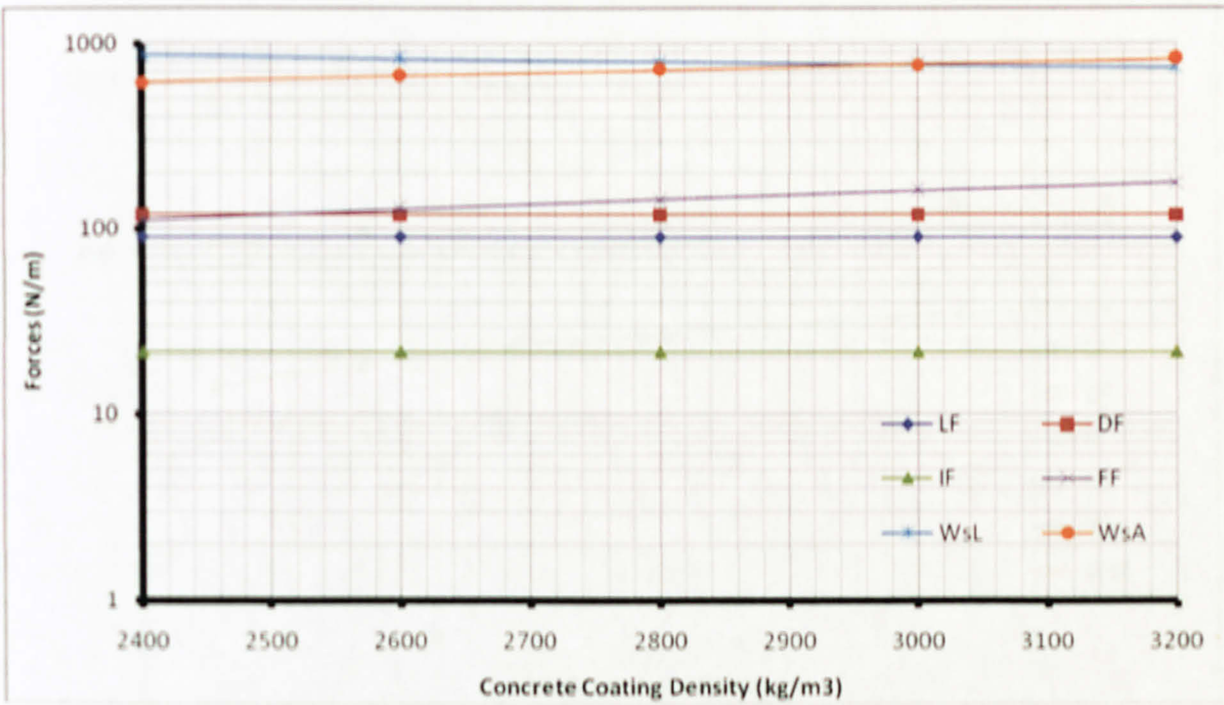
For Content Specific Gravity;



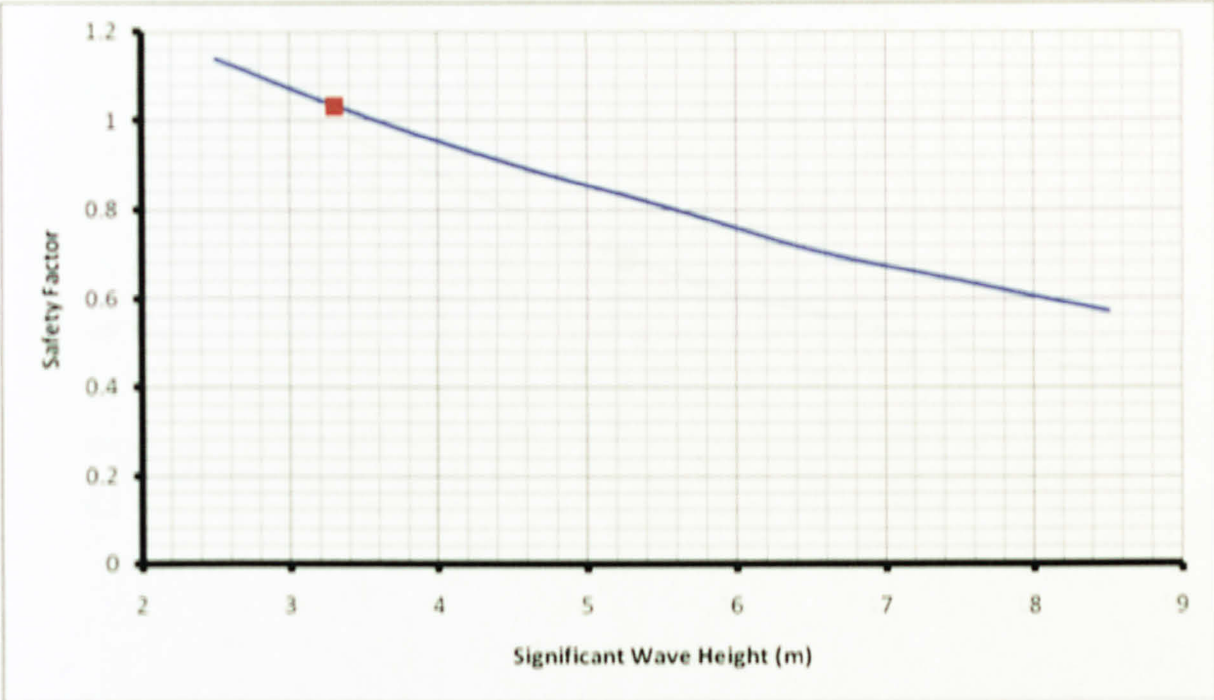


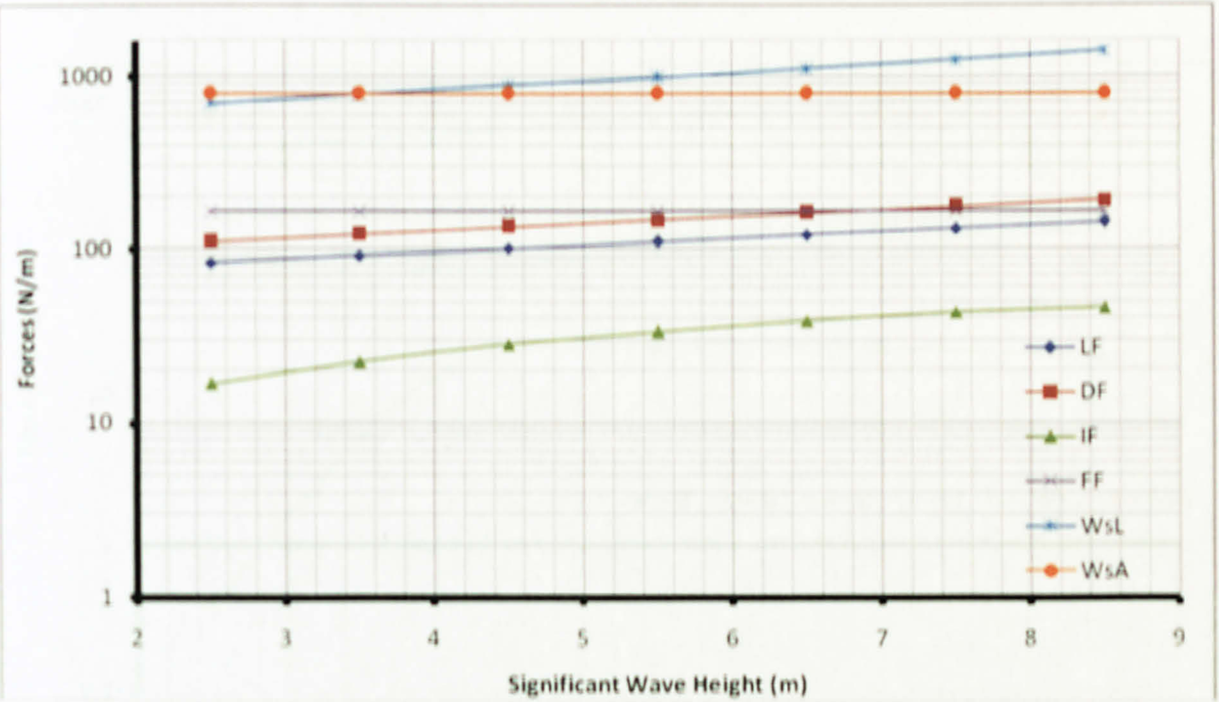
For Concrete Coating Density;



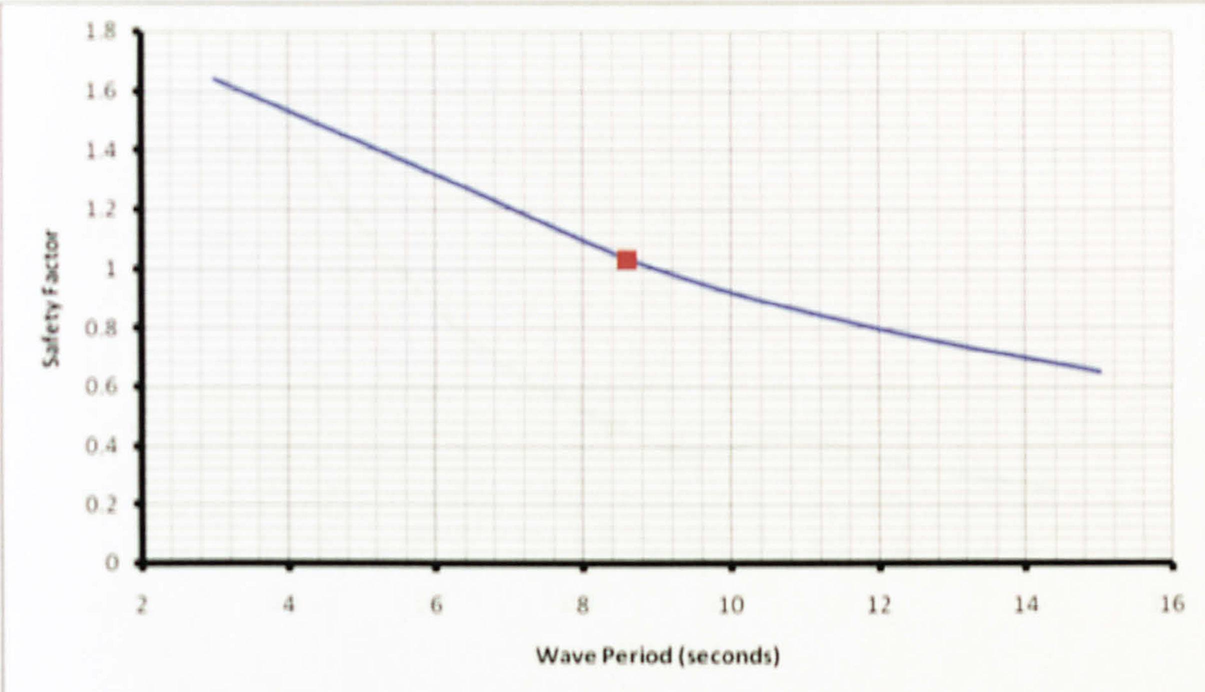


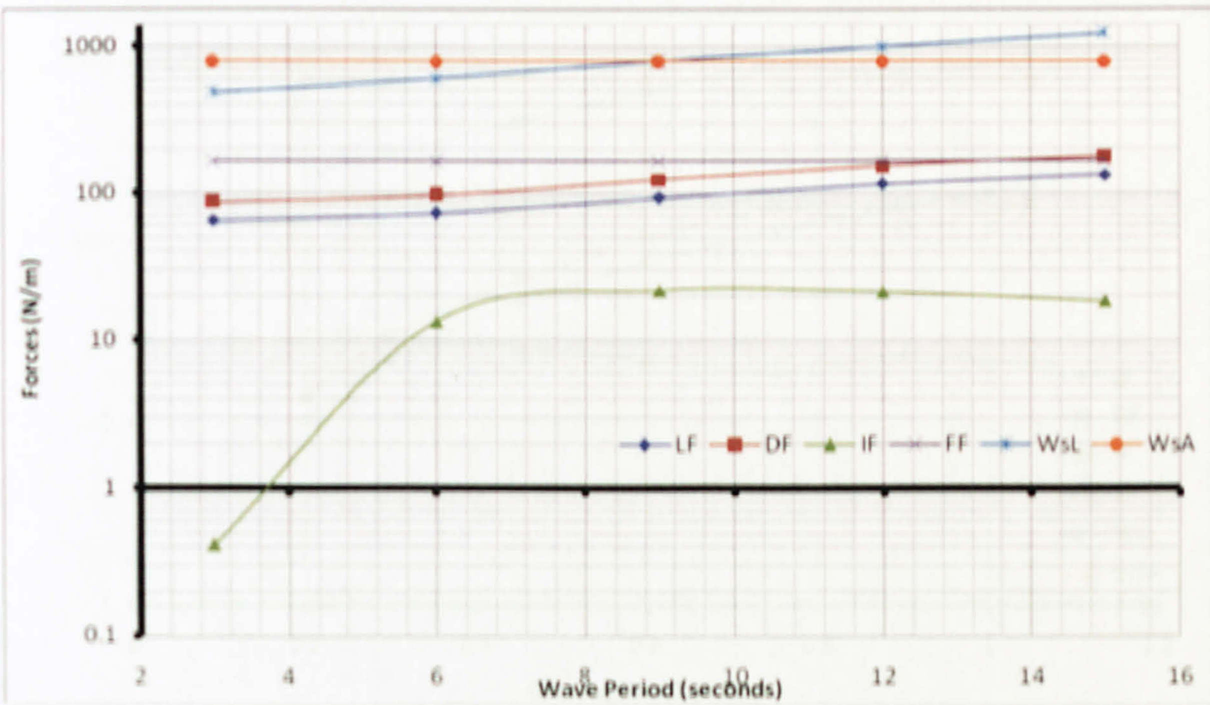
For Significant Wave Height;



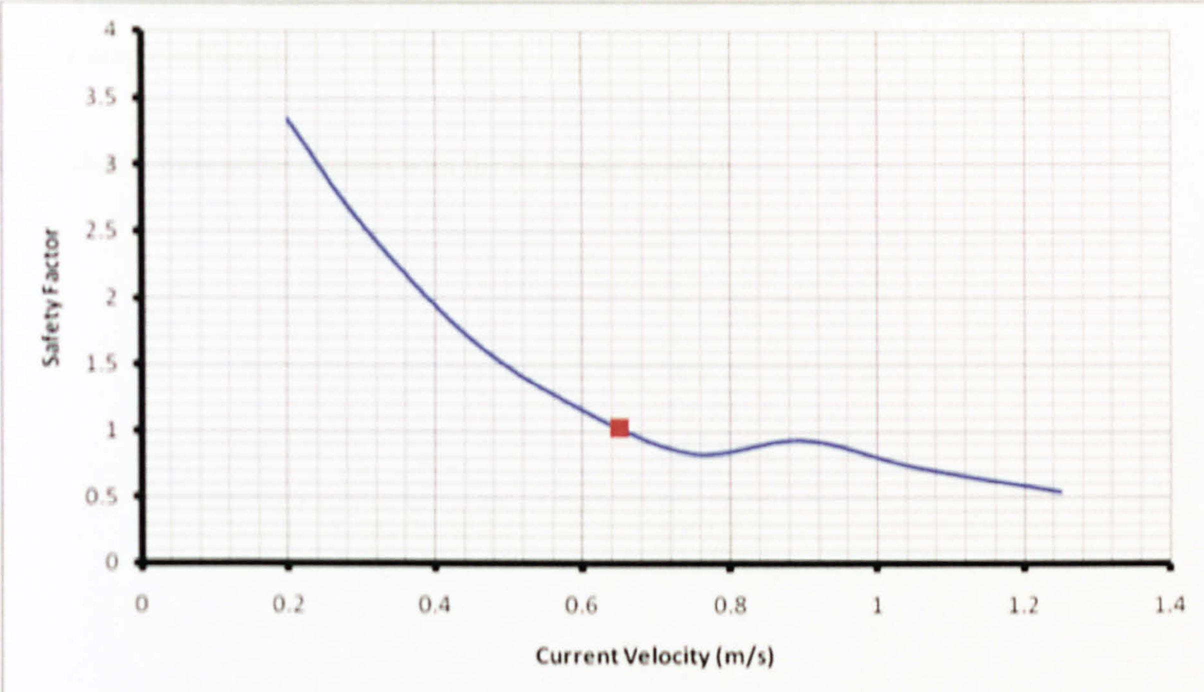


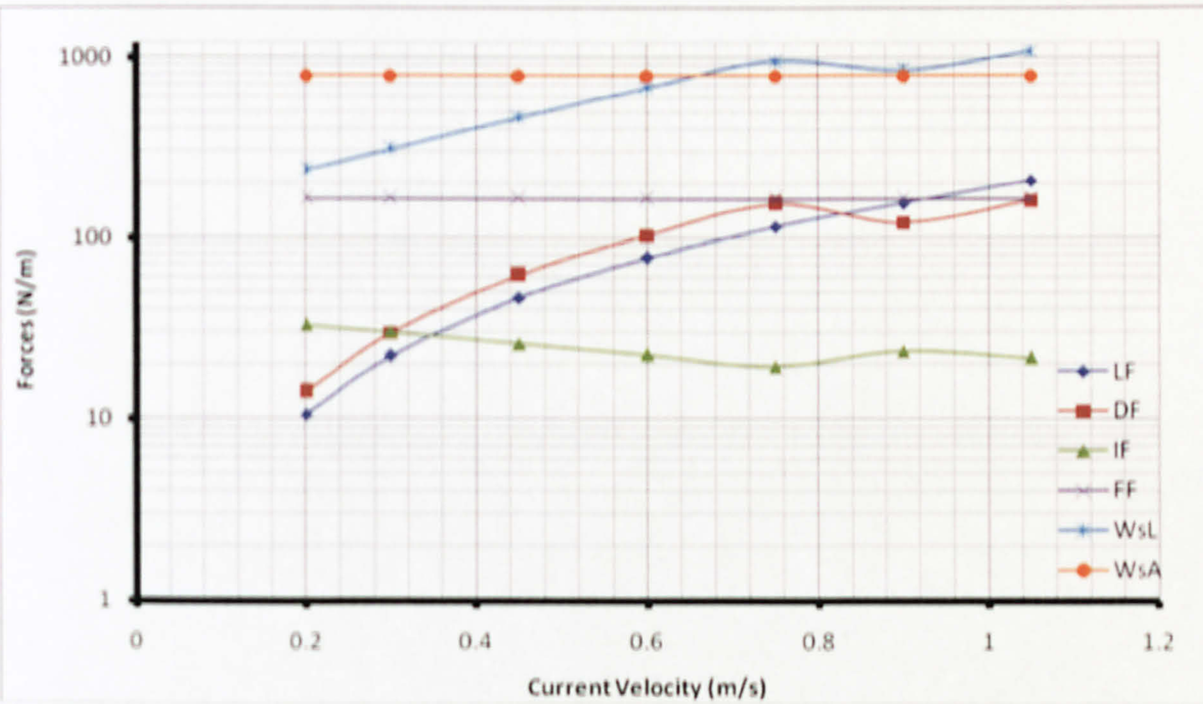
For Wave Period;





For Current Velocity;



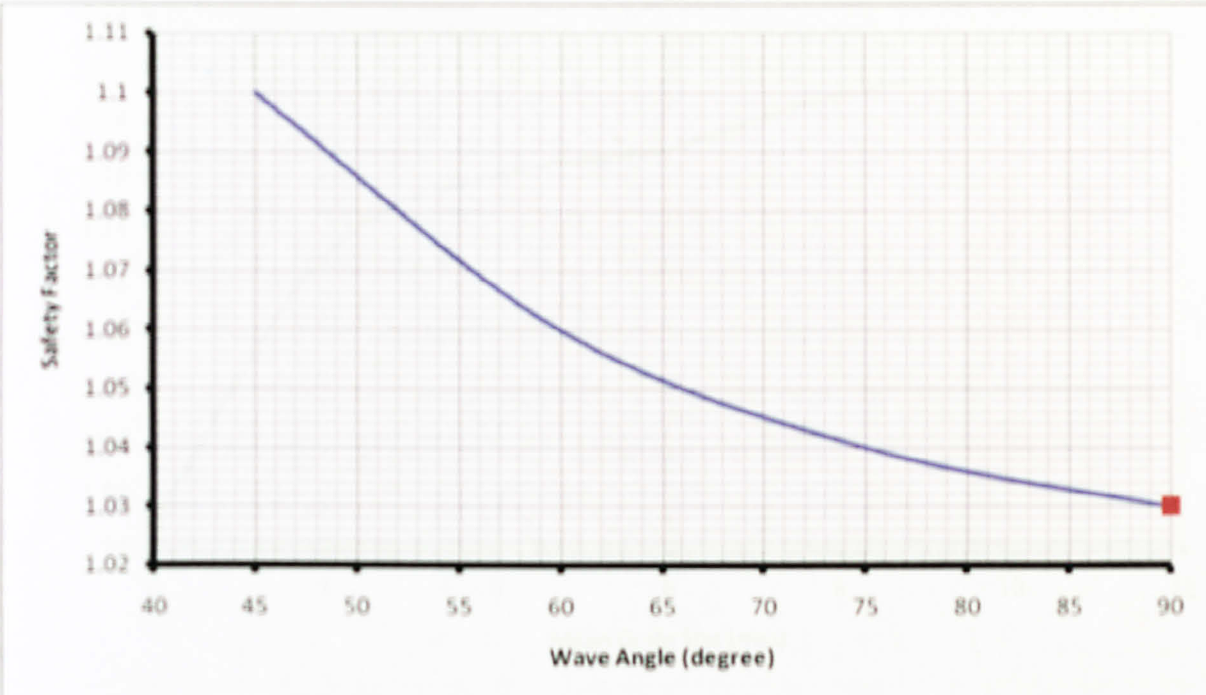


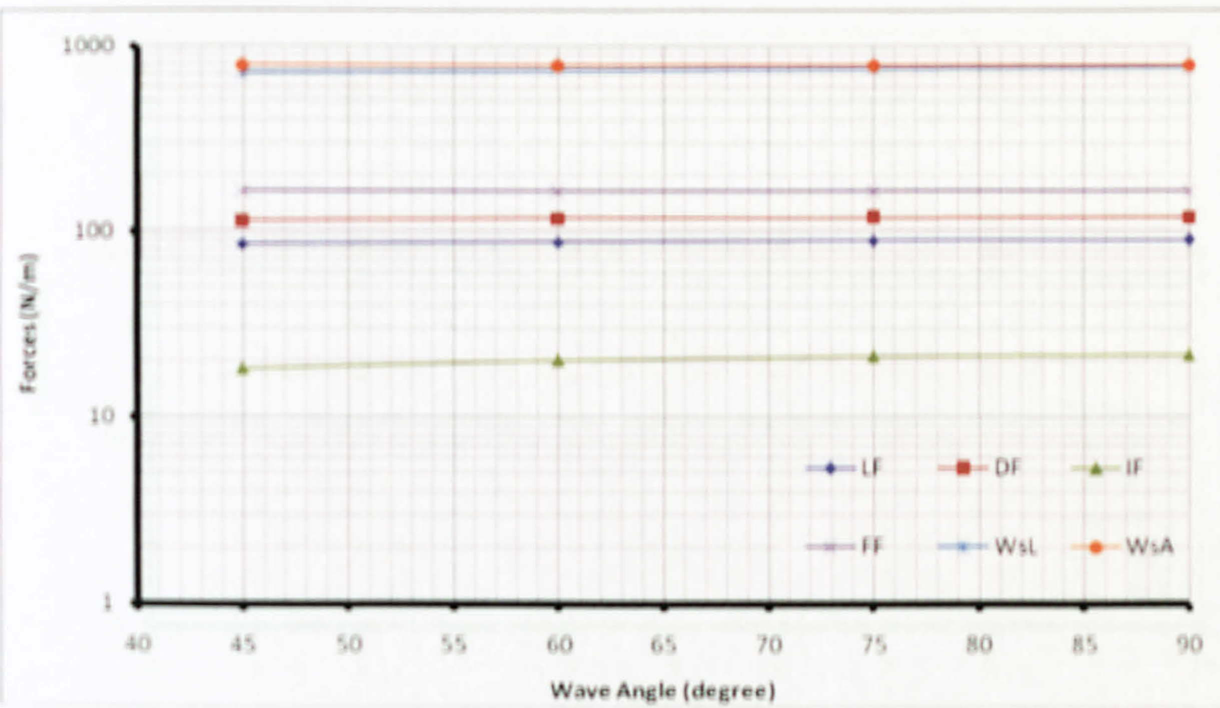
The behavior of the pipeline not as what we expected at 0.9 m/s current velocity

This happen because of reducing drag force value which due to the decreasing value of drag coefficient

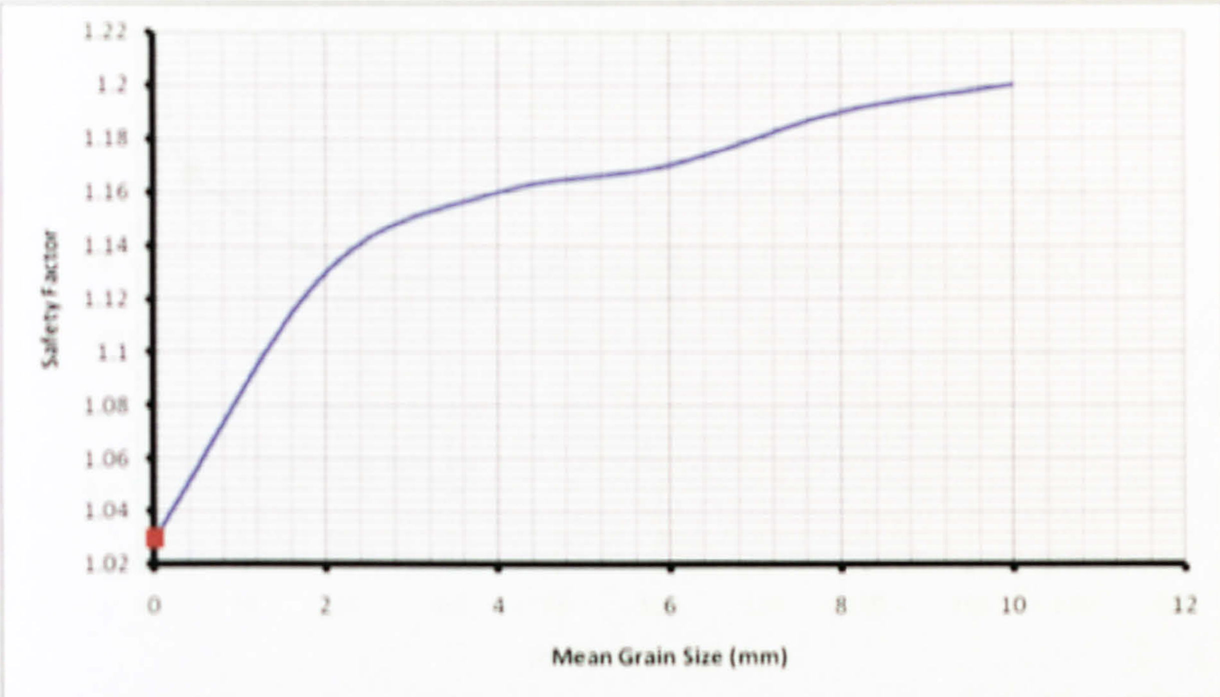
This probably have some relation with the Reynolds number.

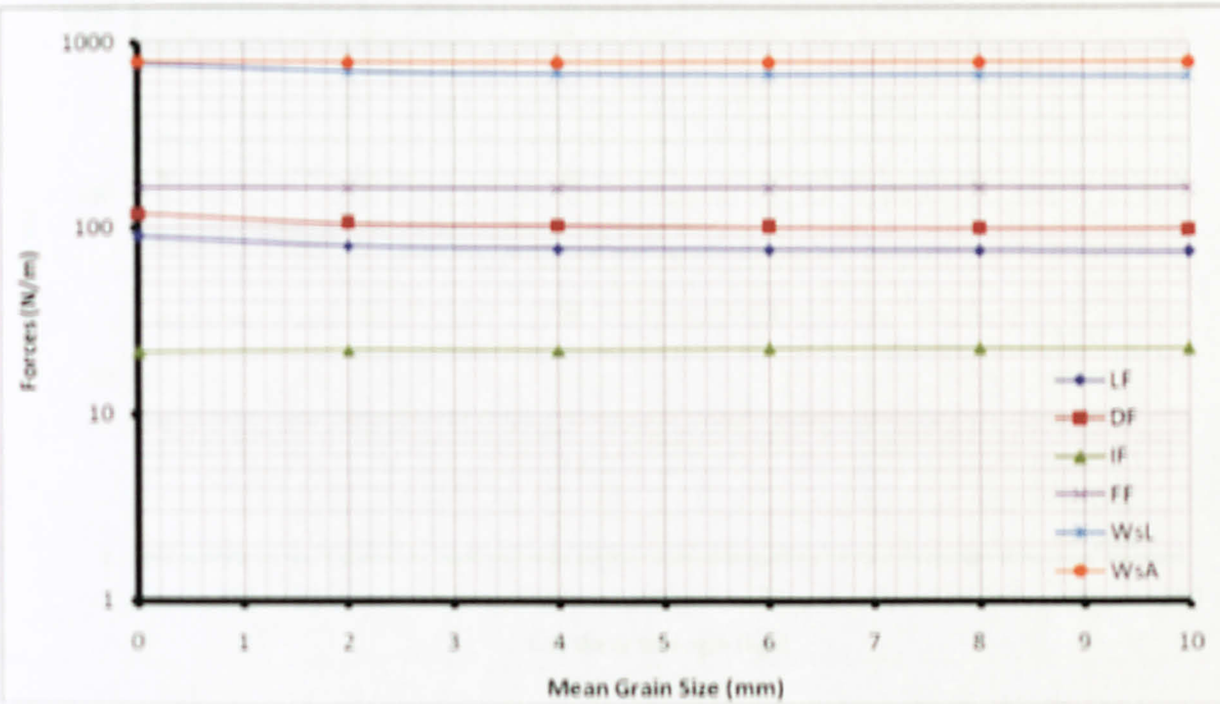
For Wave Angle;



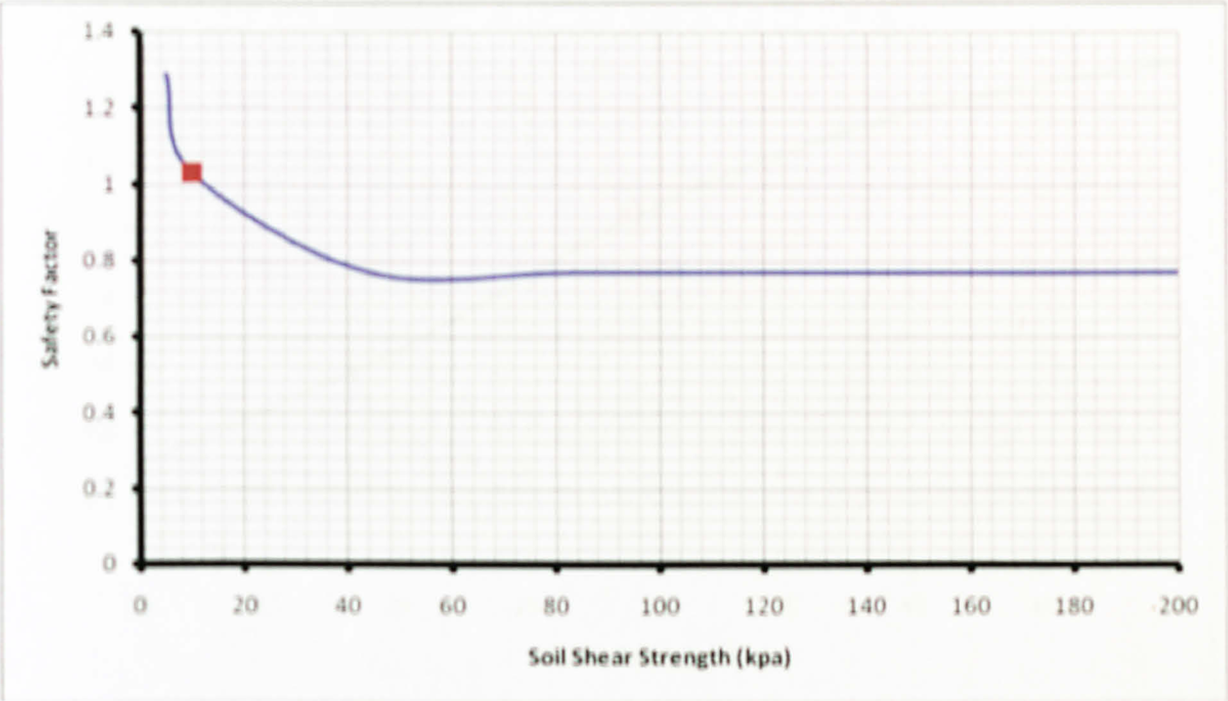


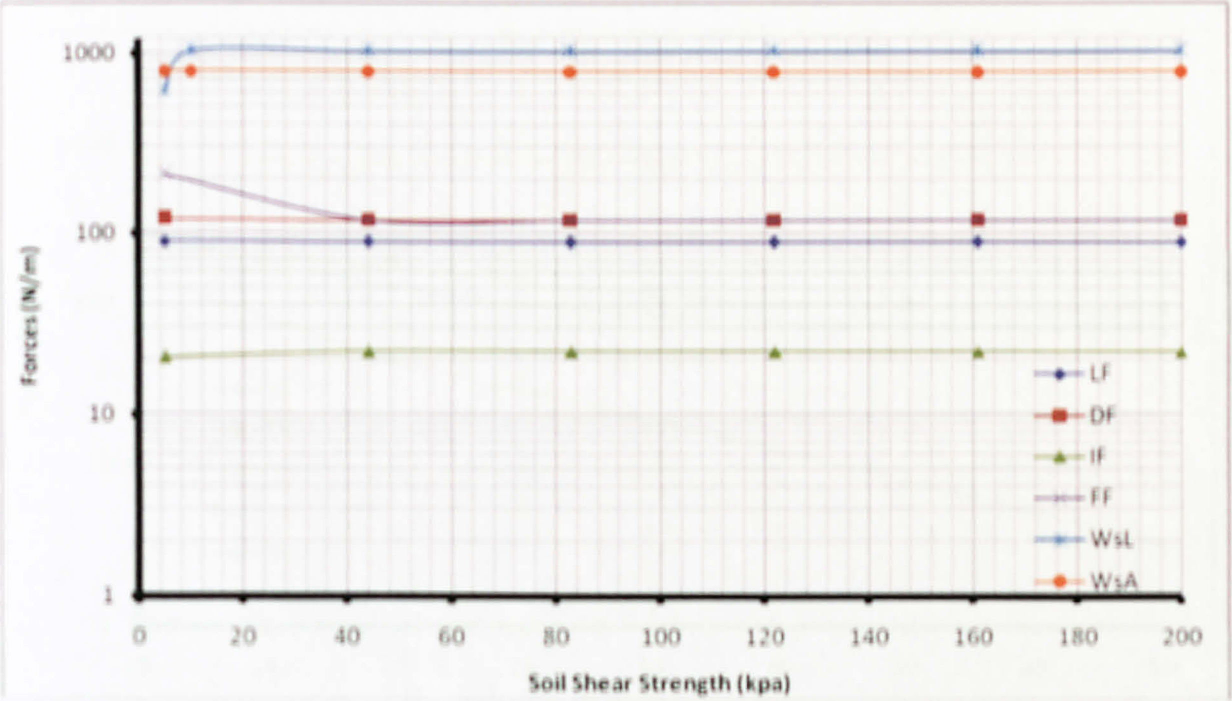
For Mean Grain Size;



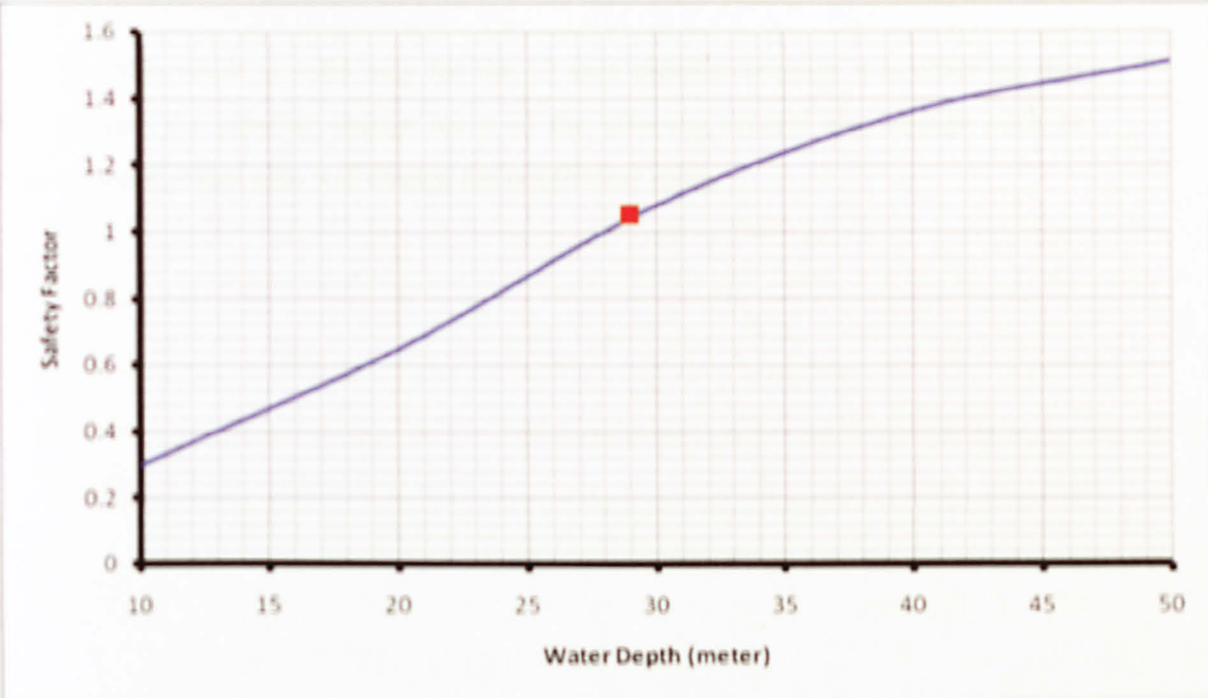


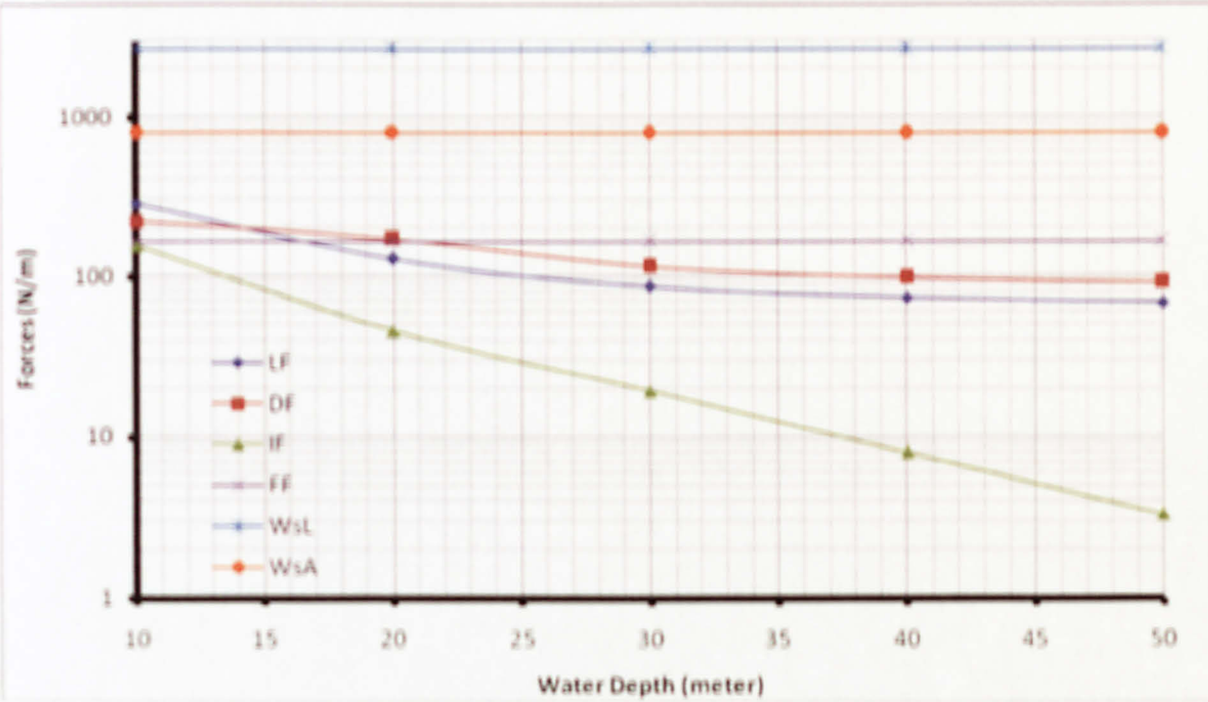
For Soil Shear Strength;





For Water Depth;





For others parameters, the behavior of the pipeline are as what we expected and no unusual result being detected.

But this result is valid only for the 12 inch pipeline because it is observed that when the size the pipeline change some of the affecting criteria also change such as Reynolds number that also will change the coefficient value.

CHAPTER 5
CONCLUSION

5.1 Conclusion

From the experiment, there are several conclusions that can be made which are regarding the mechanism of the test, outcome of the test and also regarding the parametric study.

For the mechanism of the test, it is found that it is sufficient enough to detect the force acting on the pipeline due to the environmental load. But as our pipeline model is small and the movement towards implemented load is quite severe in some cases and it is predicted that the mechanism not only measure the force by the environment but also the momentum of the pipeline model itself due to the acceleration.

The outcome of the result also not as predicted and this is due to the several conditions that cannot beings satisfied. The most important is the surface friction condition which is totally different as the test is held on the concrete surface.

The test also only being done for specific weights which are 18 kg, therefore we cannot judge how much the effect of the friction towards the outcome of the result.

Besides that, for testing the wave under the wave it is important to start with minimum wave possible such as only 8 cm wave height because as the water depth is quite small such as maximum it can be is only 1.6 meters than wave effect can be too strong and can result in damaging the mechanism especially the roller in the side guard.

For the analysis purpose in future, the industrial spreadsheet need to be re calibrated so that is can calculate the condition that implied in the lab and all the limitation that involves.

For the parametric study, some of the result is quite a new found of result as it differs from what we expected. From the result also we can see how significant the effect of the Reynolds number as it is effecting the coefficient factors.

5.2 Recommendation

For this project it is recommend that other parameters need to be change such as pipeline weight itself as it also one of the main concerns.

Soil friction also needs to be incorporated in the experiment to simulate the real condition of the submarine pipeline.

For the current and wave testing, it is good to test with same direction and different direction to see which will govern and at which water depth.

REFERENCES

- Chakrabarthi, S. K. (1994). *Offshore Structure Modeling.*

- Cokgor, S. (2002). Hydrodynamic Forces on a Partly Buried Cylinder Exposed to Combined Waves and Current. *Ocean Engineering* , 753-768.

- Finnemore, J., & Franzini, J. (2002). *Fluid Mechanics with Engineering Applications.* McGraw-Hill.

- Gao, F., Gu, X., & Jeng, D. (2002). Physical modeling of untrenched submarine pipeline instability. 1283-1304.

- Jeoung, U. C., Park, R. S., & Jo, C. H. (2002). *Flow Patterns Around Spaced Subsea Pipeline.*

- Palmer, A. C., & King, R. A. (2004). *Subsea Pipeline Engineering.*

- Sarpkaya, T., & Isaacson, M. (1981). *Mechanics of Wave Forces on Offshore Structures.* New York: Van Nostrand Reinhold.

- Sorensen, R. M. (1997). *Basic Coastal Engineering.* Chapman & Hall.

- University of Leeds, S. o. (n.d.). *Institute of Engineering Thermofluids, Surfaces and Interfaces.* Retrieved from Similarity and Modelling: http://www.cartage.org.lb/en/themes/Sciences/Physics/Mechanics/FluidMechanics/Dimensional/SimilarityModelling/SimilarityModelling.htm

- Xiaoyun, G., Fuping, G., & Qun, P. (2001). Wave-soil-pipe coupling effect on submarine pipeline on-bottom stability . *Acta Mechanica Sinica* , 86-96.

APPENDIX

RECOMMENDED PRACTICE

VOLUME E: PIPELINES AND RISERS
GROUP E 300: STRENGTH AND IN-PLACE STABILITY OF PIPELINES

RP E305

ON-BOTTOM STABILITY DESIGN OF SUBMARINE PIPELINES

OCTOBER 1988

VERITEC

Veritas Offshore Technology and Services A/S

Bennet Aprianto
Pipeline Engineer.

PREFACE

Recommended Practice are guidelines for solutions, calculation methods, technical specifications (Volume A-E) and design of offshore objects (Volume O).

The **Recommended Practice** publications cover proven technology and solutions which have been experienced by Veritec to represent good practice. The publications do not cover all areas of offshore technology, but are meant to supplement the recognized codes and standards frequently used within the industry.

The **Recommended Practice** publications are divided into 6 volumes, and each volume is divided into groups. Within each group the **Recommended Practices** are issued as selfcontained booklets. See table on next page.

Volume O gives guidelines on design of offshore objects. These publications are considered as Recommended Practices related to offshore *objects*.

Volume A-E give guidelines on specific technical solutions, methods of calculations etc. These publications are considered as Recommended Practices related to *subjects*.

RP E305 On-Bottom Stability Design of Submarine Pipelines.

- General

This **Recommended Practice** replaces the following Veritec publications,

None

- Changes in this revision of **Recommended Practice**.

None.

Table 1: General view of content of Veritec's **Recommended Practices.**

RECOMMENDED PRACTICES related to offshore *objects*

Volume O		**Design of offshore objects**
Group	O 100	Design of drilling and production facilities
Group	O 200	Design of structures
Group	O 300	Design of pipeline systems
Group	O 400	Design of subsea production systems

RECOMMENDED PRACTICES related to *subjects*

Volume A		**Quality assurance methodology**
Group	A 100	Quality systems
Group	A 200	Evaluation of contractors and suppliers
Group	A 300	Quality audits
Group	A 400	Qualification of QA/QC personnel
Group	A 500	Safety assurance systems
Group	A 600	Safety and risk analysis
Group	A 700	Documentation and information systems
Volume B		**Materials technology**
Group	B 100	Materials for structural application
Group	B 200	Materials for application in drilling, completion, production and processing systems
Group	B 300	Materials for pipelines and risers
Group	B 400	Corrosion protection
Group	B 500	Sampling and testing of materials
Group	B 600	Welding and heat treatment
Group	B 700	Non-destructive examination
Volume C		**Facilities on offshore installations**
Group	C 100	General safety
Group	C 200	Production and processing systems
Group	C 300	Instrumentation
Group	C 400	Electrical systems
Group	C 500	Drilling and well completion
Group	C 600	Mechanical equipment and piping systems
Group	C 700	Fabrication of drilling, production and processing plants
Group	C 800	Hook-up and commissioning
Group	C 900	In-service inspection and maintenance of drilling, production and processing plants.
Volume D		**Structures**
Group	D 100	Risk and reliability of structures
Group	D 200	Loads and conditions
Group	D 300	Foundation
Group	D 400	Steel structures
Group	D 500	Concrete structures
Group	D 600	Aluminium structures

Acknowledgement

This Recommended Practice for On-Bottom Stability, prepared by Veritec, has been sponsored by:

Esso Norge
Norsk Hydro
Statoil

The unlimited access to data, as well as the very valuable technical discussions/comments, from the representatives from above companies, have been highly appreciated throughout the process of preparing the RP.

SINTEF and Danish Hydraulic Institute have been subcontractors for part of the work, and we also thank them for their contributions.

We will also like to thank the members from the industry who took their time to comment and attend the review meeting on the draft version of the RP.

CONTENTS

0. LIST OF SYMBOLS

C	-	constant
D	-	nominal outside diameter of pipe
E	-	modulus of elasticity
G	-	relative soil weight, sand
K	-	Keulegan - Carpenter number, $K = U_s T_u / D$
L	-	pipe weight parameter
M	-	current to wave velocity ratio, $M = U_c / U_s$
R	-	reduction factor due to wave directionality and wave spreading
S	-	shear strength parameter
T	-	time parameter, $T = T_l / T_u$
Y	-	total displacement
A_o	-	orbital semi-diameter of particle velocity
A_s	-	significant acceleration
C_D	-	drag coefficient
C_L	-	lift coefficient
C_M	-	inertia coefficient
D_{cc}	-	outer steel pipe diam. incl. corrosion coating
D_i	-	internal pipe diameter
D_s	-	steel pipe outer diameter
F_D	-	drag force
F_I	-	inertia force
F_L	-	lift force
F_w	-	load factor
H_s	-	significant wave height
K_b	-	equivalent sand roughness parameter
S_f	-	safety factor
$S_{\eta\eta}(\omega)$	-	wave spectrum (long-crested sea)
$S_{uu}(\omega)$	-	near-bottom horizontal velocity spectrum
S_u	-	undrained shear strength of clay soil
T_l	-	time length
T_n	-	parameter, $T_n = \sqrt{(d/g)}$
T_p	-	spectral peak period of surface wave spectrum
T_u	-	mean zero up-crossing period
U_c	-	current velocity perpendicular to the pipe
U_s	-	significant velocity perpendicular to the pipe
U_s^*	-	significant velocity perpendicular to the pipe (no reduction factor included).
U^*	-	friction velocity
U_D	-	average velocity over pipe diameter, D
U_r	-	reference steady velocity
$U(z)$	-	steady flow velocity
W_s	-	submerged pipe weight
W_{sd}	-	design weight
d	-	water depth
d_{50}	-	mean grain size
f_{ws}	-	correction factor on submerged weight
g	-	gravity constant
k	-	wave number
m_o, m_2	-	spectral moments
n	-	spreading exponent
t_s	-	steel pipe thickness
z	-	elevation above seabed
z_o	-	bottom roughness parameter
z_{oa}	-	apparent roughness

z_r	-	reference height above seabed
α	-	Phillips' constant
β	-	sub-direction around main wave direction
δ	-	scaled lateral displacement
$\varepsilon, \varepsilon'$	-	engineering and generalized strain
$\psi(\beta, \theta)$	-	spreading function
γ	-	peakedness parameter in Jonswap wave spectrum
κ	-	von Karman's constant
μ	-	soil friction factor
ω	-	angular frequency
ω_p	-	angular frequency of spectral peak
ρ_c	-	density of concrete coating
ρ_{cc}	-	density of corrosion coating
ρ_i	-	density of internal content
ρ_s	-	density of sand soil
ρ_{st}	-	density of steel material
ρ_w	-	mass density of water
σ	-	spectral width parameter
θ	-	main wave direction, phase angle
θ_p	-	direction perpendicular to the pipeline

1. INTRODUCTION

This Recommended Practice outlines the basic considerations with regard to the stability design of submarine pipelines.

The main objectives of this recommended practice are to make the latest state-of-the-art information on pipeline stability available for use in the design of submarine pipelines, and to provide a framework from which stability design methods can be developed further as more information becomes available. The RP is mainly based on the results from the Pipeline Stability project PIPESTAB carried out by SINTEF (1983 - 1987) and sponsored by Esso Norge A/S and Statoil, see /2/ - /8/.

Results from other research programs may be equally applicable for On-Bottom Stability of Pipelines. It is the intention, through revisions of the present RP, to incorporate other results/data as they become available and thereby extend the limits for use.

The design method presented in this Recommended Practice relates to a pipeline resting on the sea bed throughout its lifetime, or prior to some other form of stabilization (eg. trenching, burial, self-burial). The stability of the pipeline is then directly related to the submerged weight of the pipeline, the environmental forces and the resistance developed by the sea bed soil. Consequently the aim of stability design is to verify that the submerged weight of the pipeline is sufficient to meet the required stability criteria.

2. DESIGN CONDITIONS

2.1 Basic Conditions

2.1.1 The following basic conditions should be considered during the on-bottom stability design of submarine pipelines:

- Environmental conditions
- Geotechnical conditions of the sea bed
- Topographical conditions of the sea bed (eg. slope, rock outcrops, depressions)
- Bathymetry (water depth)
- Pipe data (diameter, wall thickness, concrete coating)
- Location of pipeline restraints (riser connections, crossings, etc)

2.2 Return Periods

2.2.1 The stability design is to be based on a given return period of near-bottom environmental conditions acting perpendicular to the pipe. In general, both near-bottom wave induced particle velocities and near-bottom currents will need to be considered.

2.2.2 If sufficient information is available on joint probability of waves and current, then the combined wave and steady current with 100 year recurrence interval should be used. If inadequate information is available on the joint probability of waves and current, then the following are suggested for the operational condition:

If waves dominates forces	Waves :	100 year return condition of near-bottom wave-induced particle velocity normal to the pipeline.
	Current :	10 year return condition.
If current dominates forces	Waves :	10 year return condition
	Current :	100 year return condition.

2.2.3 For temporary phases, the recurrence period should be taken as follows:
Duration less than 3 days: The environmental parameters for determination of environmental loads may be established based on reliable weather forcasts.
Duration in excess of 3 days: a) No danger for loss of human lives. A return period of 1 year for the relevant season can be applied.
b) Danger for loss of human lives. The environmental parameters should be defined with a 100 year seasonal return period.
However, the relevant season should not be taken less than 2 months.

2.3 Environmental Conditions

2.3.1 The following environmental conditions should be evaluated at a number of positions along the length of the pipeline :

- Waves
- Currents

The number of positions necessary to adequately define the environment will be dependent on the length of the pipeline and the variations in water depth, seabed soil and meteorological conditions.

2.3.2 The environmental conditions used in the stability design should be based on adequate data from the area in question. The data may be from measurements, hindcast models, or visual observations. If sufficient data on the particular area is not available, reasonably conservative estimates based on data from other nearby locations may be used.

2.3.3 Recognised methods of statistical analysis should be used to describe the random nature of the environmental conditions. Seastates will normally be defined in terms of the significant wave height (H_s), spectral peak period (T_P) and corresponding return probability.

2.3.4 The form in which the wave information is available, is dependent on the amount and quality of data available for the particular location in question. This may range from a joint distribution of H_s and T_P with directional information to an omnidirectional design value for H_s with an estimated period. The design method presented in section 3. will accept wave input of varying degrees of sophistication.

2.3.5 The peak period (T_P) will depend on fetch and depth limitations as well as duration of the seastates. If no other information is available for the peak period, then the following relationship may be used for the upper limit:

$$T_P = \sqrt{(250\, H_s/g)}$$

If a joint distribution of H_s and T_P is available, then the combination of H_s - T_P which gives the most extreme near-bottom conditions should be selected.

2.3.6 The directional distribution of the wave conditions may be accounted for when selecting the design wave-induced particle velocity. Normally extreme seastates from different directions will need to be considered. If no directional wave information is available then the extreme wave conditions should be assumed to act perpendicular to the axis of the pipeline.

2.3.7 The short crestedness of the waves may be accounted for when selecting the design wave-induced particle velocity. If no site specific information is available, then this may be taken into account by consideration of the energy spreading away from the main direction of wave propagation.

2.3.8 The wave-induced particle velocity to be used in the stability design analysis is represented by the significant value of the near-bottom velocity perpendicular to the pipeline (U_s), and the corresponding mean zero up-crossing period (T_u).

2.3.9 When calculating U_s and T_u, the most appropriate formulation for the water surface elevation spectrum should be used. For North Sea conditions the Jonswap spectral formulation is recommended. For long-crested seas, the Jonswap spectrum is given by:

$$S_{\eta\eta}(\omega) = \alpha g^2 (\omega)^{-5} \exp\left(-5/4(\omega/\omega_p)^{-4}\right) \gamma^a$$ — FOR LONG CRESTED SEAS

$$a = \exp\left\{\frac{-(\omega-\omega_p)^2}{2\sigma^2\omega_p^2}\right\}$$

where

ω	=	angular frequency
ω_p	=	angular frequency of spectral peak
g	=	acceleration due to gravity
α	=	Phillips' constant
σ	=	spectral width parameter
		$\sigma = 0.07$ if $w \le \omega_p$
		$\sigma = 0.09$ if $w > \omega_p$
γ	=	peakedness parameter

2.3.10 U_s and T_u may be calculated by transforming the long-crested water surface elevation spectrum to the bottom and applying a reduction factor to account for wave directionality with respect to the pipe and for short-crestedness of the waves as follows:

$$S_{uu}(\omega) = (\omega / \sinh(kd))^2 \cdot S_{\eta\eta}(\omega)$$

where

$S_{\eta\eta}(\omega)$	=	water surface elevation spectrum (long-crested)
k	=	wave number ($\omega^2 = gk \tanh(kd)$)
ω	=	circular frequency

and

$$U_s = U_s * R$$

$$T_u = 2\pi(m_0/m_2)^{\frac{1}{2}}$$

where

$$U_s^* = 2\sqrt{m_0}$$

$$m_n = \int_0^\infty \omega^n S_{uu}(\omega)\cdot d\omega$$

R = reduction factor

U_s^* and T_u may be obtained through the non-dimensional curves presented in Figs. 2.1 and 2.2. The suitability of first order wave theory when approaching shallow water should be verified.

The reduction factor due to wave directionality and wave spreading given by a cosn function, is given by:

$$R = \left\{ \int_{\theta-\pi/2}^{\theta+\pi/2} \psi(\beta,\theta)\left(\cos^2(\theta_p-\beta)\right)d\beta \right\}^{\frac{1}{2}}$$

where

θ_p = direction perpendicular to the pipeline
θ = main wave direction
β = sub-direction around the main wave direction
$\psi(\beta,\theta)$ = spreading function, given by :

$$\psi(\beta,\theta) = C\cos^n(\beta-\theta)$$

n = spreading exponent (site specific)
C = constant chosen such that the integral of R over all wave directions is equal to 1.0

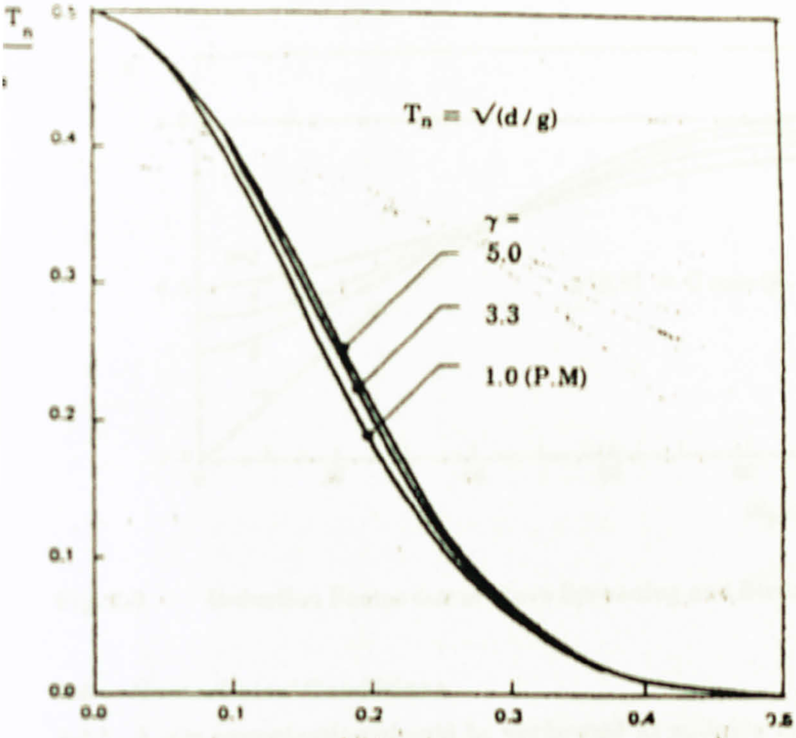
R may be obtained from Fig. 2.3.

2.3.11 The design current velocities should be based on a consideration of the various contributing components such as tidal, storm surge and circulation currents.

2.3.12 The directional distribution of the current velocity may be used in the stability design. If no such information is available, the current should be assumed to act perpendicular to the axis of pipeline.

2.3.13 The current velocity may be reduced to take account of the effect of the bottom boundary layer. This may be achieved using a suitable boundary layer model. The velocity profile in the boundary layer should be integrated over the pipeline diameter to give an effective current velocity. An approximate method of estimating a boundary layer reduction is presented in Appendix A.

2.3.14 It is not recommended to consider any boundary layer effect on wave induced velocities as such effects are normally small and are implicitly included in the applied hydrodynamic force model, which is the basis for the generalized curves. However, the effect of waves on the current boundary layer may be estimated as shown in Appendix A. In special cases of very small diameter pipes where the wave boundary layer may be important, further velocity reductions should be justified by relevant data.



d = water depth
g = gravity constant
γ = Jonswap, peakedness parameter
P.M = Pierson Moskovitz

Fig. 2.1 Significant Water Velocity, U_s*
(Linear Wave Theory. Wave Directionality and Spreading not accounted for)

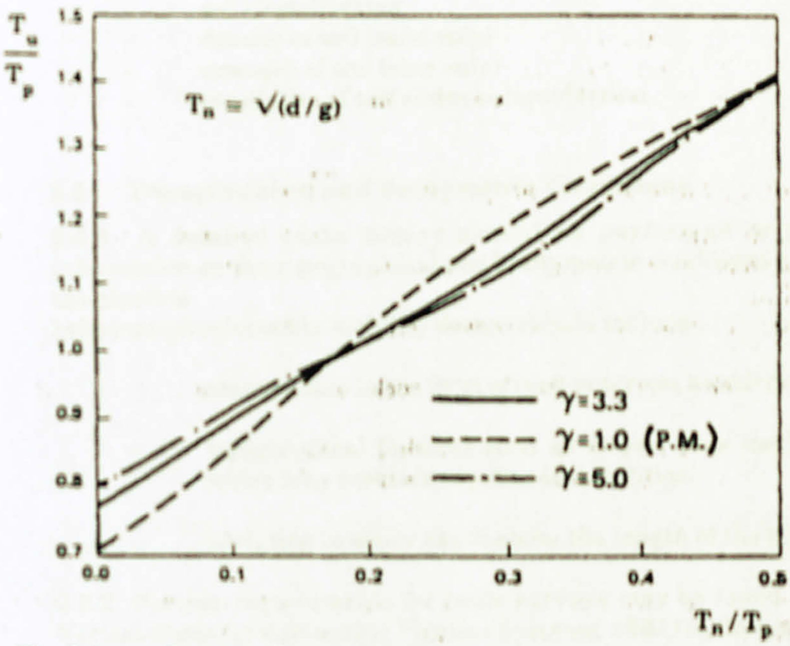


Fig. 2.2 Zero-Up-Crossing Period, T_u

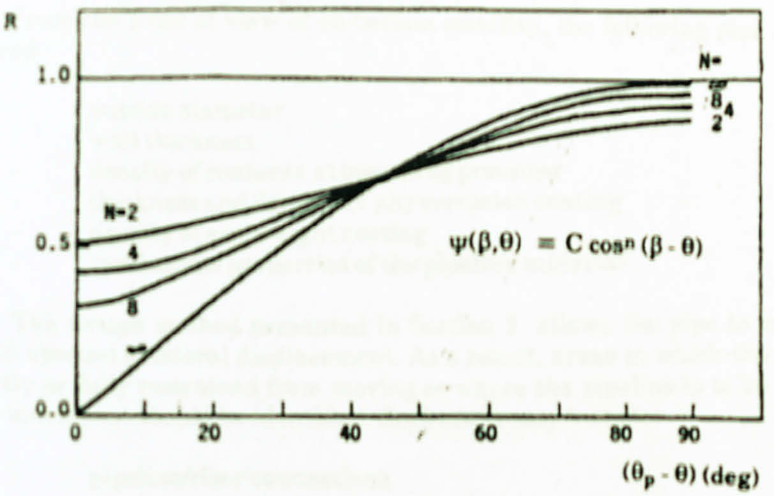


Fig. 2.3 Reduction Factor due to Wave Spreading and Directionality

2.4 Geotechnical Conditions

2.4.1 A site investigation should be performed at suitable intervals along the route of the pipeline. The number of intervals will be dependent on the length of the pipeline and the anticipated variations in geotechnical conditions. Suitable sampling techniques should be used during the site investigation.

Guidelines for site and laboratory testing may be found in Veritas' RP D301 /9/.

2.4.2 From the point of view of stability design the site investigation should provide the following information for the sea bed soil on and immediately below the surface of the sea bed :

- soil classification
- density of soil (sand only)
- strength of soil (clay only)
- possibility of soil slides or liquefaction.

2.5 Topographical and Bathymetric Conditions

2.5.1 A detailed route survey should be performed to provide suitable information on the topographical and bathymetric conditions along the length of the pipeline.
Information relevant to stability design should include :

- obstructions in the form of rock outcrops, boulders or wrecks

- topographical features such as slopes, pock marks or other items which may result in pipeline instabilities.

- variation in water depth along the length of the pipeline.

2.5.2 Further requirements for route surveys may be found in the Det norske Veritas Rules for Submarine Pipeline Systems, 1981 /1/, Sections 2.2.2 and 2.2.3.

2.6 Pipe Data

2.6.1 From the point of view of on-bottom stability, the following pipe data are required :

- outside diameter
- wall thickness
- density of contents at operating pressure
- thickness and density of any corrosion coating
- density of any weight coating
- mechanical properties of the pipeline material

2.6.2 The design method presented in Section 3. allows the pipe to undergo a certain amount of lateral displacement. As a result, areas in which the pipe-line is partly or fully restrained from moving or where the pipeline is to be designed for no movement should be identified. Such areas may include:

- pipeline/riser connections
- pipeline crossings
- subsea valves
- expansion loops
- pipeline emerging from a trench

3. DESIGN METHOD

3.1 General

3.1.1 The design method presented in this section relates to a pipeline resting on the sea bed throughout its' lifetime or prior to some other form of stabilization (eg. trenching, burial, mattresses or other point stabilisation). The stability of the pipeline is then directly related to the submerged weight of the pipeline, the environmental forces and the resistance developed by the sea bed soil. Consequently the aim of the stability design is to verify that the submerged weight of the pipeline is sufficient to meet the required stability criteria.

3.1.2 The following design criteria should be considered during the stability design :

- lateral displacement
- stress/strain in pipe wall
- interaction with lateral buckling due to axial forces
- fatigue damage
- wear and deterioration of the coating
- damage to sacrificial anodes

In general the lateral displacement and the stress/strain experienced by the pipeline will be the governing design criteria. Further consideration of the design criteria is presented in section 4.

3.2 Load Cases

3.2.1 All load cases relevant to the stability of the pipeline should be considered. In general this will normally result in two load cases namely :

- Installation Condition
- Operating Condition

3.2.2 The Installation Condition relates to the period of time after installation when the pipeline is resting on the sea bed prior to trenching or commissioning. Unless the pipeline will be water flooded immediately upon installation, the pipeline should normally be assumed air filled during this condition. For a pipeline which is to be trenched, the installation condition will normally determine the pipeline submerged weight requirements. For the Installation Condition, a minimum specific gravity $(W_s + B) / B = 1.1$ is required (W_s = submerged weight, B = buoyancy).
In general, a water absorption of 5% of concrete weight can be included.

3.2.3 Details of the design storm conditions related to the installation phase are given in 2.2.3.

3.2.4 The Operating Condition relates to the operating phases of the pipeline lifetime. In the stability analysis, the pipeline should be assumed to be filled with contents at normal operating pressure and expected lowest density.

3.2.5 During the operating condition the pipeline may be subjected to lateral displacements, stresses/strains etc. due to extreme wave and current conditions, however the pipeline should still remain serviceable after the storm situation. The design combination of extreme wave and current should be determined so that its exceedance probability does not exceed 10^{-2}/year (100 year return period).

3.2.6 Details of the environmental conditions to be applied during the operational condition are given in 2.2.2.

3.3 Analysis Methods

3.3.1 There are several analysis methods available on which to base pipeline stability design. Three different methods are considered in this Recommended Practice, namely:

- (i) Dynamic Analysis
- (ii) Generalized Stability Analysis
- (iii) Simplified Stability Analysis

The choice of the above analysis methods is dependent on the degree of detail required in results of the design analysis.

3.3.2 Dynamic Analysis involves a full dynamic simulation of a pipeline resting on the seabed, including modelling of soil resistance, hydrodynamic forces, boundary conditions and dynamic response. Dynamic analysis forms the reference base for the generalized method. It may be used for detailed analysis of critical areas along a pipeline, such as pipeline crossings, riser connections etc., where a high level of detail is required on pipeline response, or for reanalysis of a critical existing line.

3.3.3 The Generalized Stability Analysis is based on a set of non-dimensional stability curves which have been derived from a series of runs with a dynamic response model. This method can be used in either detailed design calculations or preliminary design calculations. The Generalized Stability Analysis method may be used on the sections of the pipeline where potential pipeline movement and strain may be important. The main assumptions of the method are given in section 5.2.

3.3.4 The Simplified Stability Analysis is based on a quasi-static balance of forces acting on the pipe, but has been calibrated with results from the generalized stability analysis. The method generally gives pipe weights that form a conservative envelope of those obtained from the generalized stability analysis.

This method may be used for the vast majority of stability calculations, where the required submerged weight is the only parameter of interest. The method is based on simplified models, consequently it is recommended that this method should not be modified in any way without a full consideration of all the relevant factors, i.e. checking with one of the above two analysis methods.

3.3.5 If the partial burial of a pipeline (implying stable pipe) is to be taken account of in the stability design, then the following should be considered in the stability calculations :

- method to be based on static considerations only, i.e. the pipe should not break out, i.e. be pulled out of the partially buried condition.
- the most probable maximum 100 year near-bottom wave-induced velocity and acceleration normal to the pipeline should be used in the calculation of hydrodynamic forces.
- realistic hydrodynamic force models should be used.
- a soil resistance model which realistically represents the pipe-soil interaction should be used.

3.4 Sinking/Floatation

3.4.1 Buried lines should be checked for possible sinking or floatation. For both liquid and gas lines, sinking should be considered assuming the pipe to be water filled and floatation should be considered assuming the pipe to be gas or air filled.

3.4.2 If the specific weight of the water filled pipe is less than that of the soil (including water contents), no further analysis is required to document the safety against sinking. For lines to be placed in soils having low shear strength, a consideration of the soil stress may be necessary. If the soil is, or is likely to be liquified, the depth of sinking should be limited to a satisfactory value, by consideration of the depth of liquifaction or the build up of resistance during sinking.

3.4.3 If the specific gravity of the gas or air filled pipe is less than that of the soil, the shear strength of the soil should be documented as being sufficient to prevent floatation. Consequently, in soils which are or may be liquified, the specific weight of the gas or air filled pipe should not be less than that of the soil (if burial is required).

3.4.4 Exposed lines resting directly on the sea bed should be checked for possible sinking in the same manner as explained for buried lines, in section 3.4.2 above.

3.5 Overview of the Design Method

3.5.1 The flow diagram presented in Figure 3.1 shows an overview of the design method outlined above.

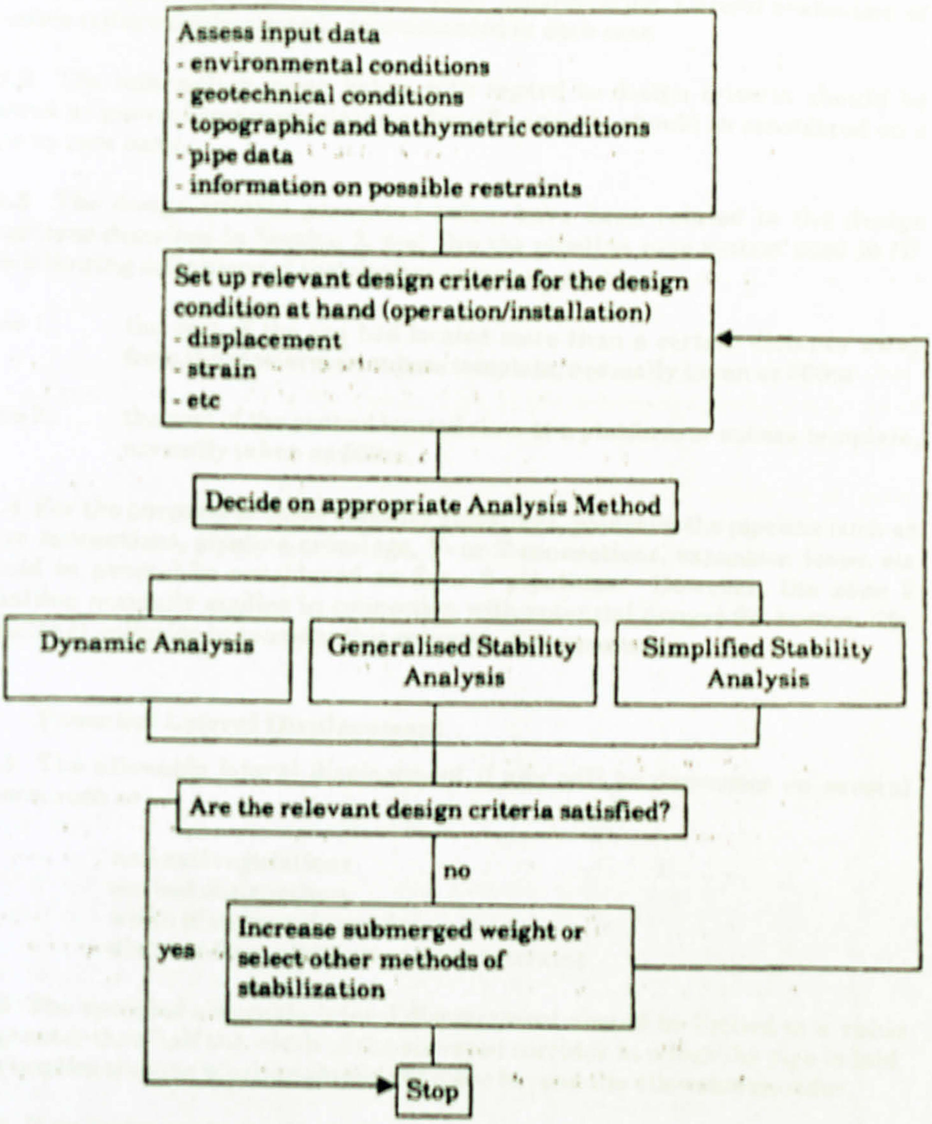


Figure 3.1 Overview of Design Method

4. DESIGN CRITERIA

4.1 General

4.1.1 The criteria to be used in the stability design method outlined in Section 3, will vary depending on the situation under consideration. Careful evaluation of possible failure mechanisms is recommended in each case.

4.1.2 The information given below with regard to design criteria should be viewed as general recommendations. Specific criteria should be considered on a case by case basis.

4.1.3 The design criteria presented below have been related to the design conditions described in Section 3, and also the pipeline zone system used in /1/. The following definitions of the pipeline zones are used :

Zone 1 : the part of the sea bed located more than a certain distance away from the platform or subsea template, normally taken as 500m

Zone 2 : the part of the seabed located close to a platform or subsea template, normally taken as 500m.

4.1.4 For the purposes of these stability guidelines, points on the pipeline such as valve connections, pipeline crossings, Y- or T-connections, expansion loops, etc should in general be considered as Zone 2 pipelines. However, the zone 2 definition normally applies in connection with potential danger for human life, significant pollution or considerable economic consequences.

4.2 Potential Lateral Displacement

4.2.1 The allowable lateral displacement if any will be dependent on several factors, such as :

- national regulations
- sea bed obstructions
- width of surveyed corridor
- distance from platform or other restraint

4.2.2 The specified allowable lateral displacement should be limited to a value not greater than half the width of the surveyed corridor in which the pipe is laid. This implies that the pipeline should not move beyond the allowable corridor.

4.2.3 If no further information is available, then the following may be used for the allowable maximum lateral displacement in the operational condition :

Zone 1	20 m
Zone 2	0 m

This criteria can be relaxed if other relevant data are available.The pipe must also be able to satisfy the other relevant design criteria at the above allowable displacement. For most situations the lateral displacement will be the governing criteria. In general, the strain requirement will also be satisfied when limiting the movement to maximum 20 m. The sensitivity to variations in environmental parameters (wave height/period) should be checked. The allowable displacement criteria refer to a seastate duration of 3 hours at maximum storm intensity.

4.2.4 For Zone 2 pipelines the allowable lateral displacement may be increased above zero if the effect of the displacement can be acceptably accommodated by the pipeline and the supporting structure (eg. riser connection)

4.2.5 The allowable lateral displacement for the installation condition is dependent on the time period between laying and commissioning, and should be decided on a case by case basis. However, if the recommendations with respect to environmental conditions given in 2.2.3 are followed an allowable displacement of 5 m is suggested.

4.3 Bending Strain

4.3.1 Due to the development of bending moments at points of fixity along the pipeline, as a result of the lateral displacement, the bending strains experienced by the pipe should be evaluated during the stability design.

4.3.2 For known points of fixity, such as riser connections, subsea valves, subsea templates etc, the effect of the lateral pipe displacement should be evaluated for both the pipeline and the restraining structure.

4.3.3 Any part of the pipeline may bend as a result of local variations in seabed and pipe properties, and the bending strain criterion should be satisfied at any point assuming a fixed end restraint. This applies to the generalized method.

4.3.4 When evaluating the bending effects resulting from the lateral displacement of the pipeline, consideration should be given to the following:

- excessive straining
- ovalization
- buckling

Reference is made to /1/ for the limiting criteria for the above.

4.3.5 If no further information is available, the limiting strain criteria may be taken as $7.5/(D/t)^2$, with a maximum strain limit of 1 percent, see Fig. 4.1. The limiting strain values relate to total (static + dynamic) accumulated elasto/plastic strain, not elastic strain. Consequently, when using this strain criteria the ductility of the pipeline material should be taken into account. The limiting strain values may only be used if a full dynamic analysis applying nonlinear elasto/plastic elements is used. If nonlinear strain is used in design some check of pipe behaviour in the ductility level events is required.

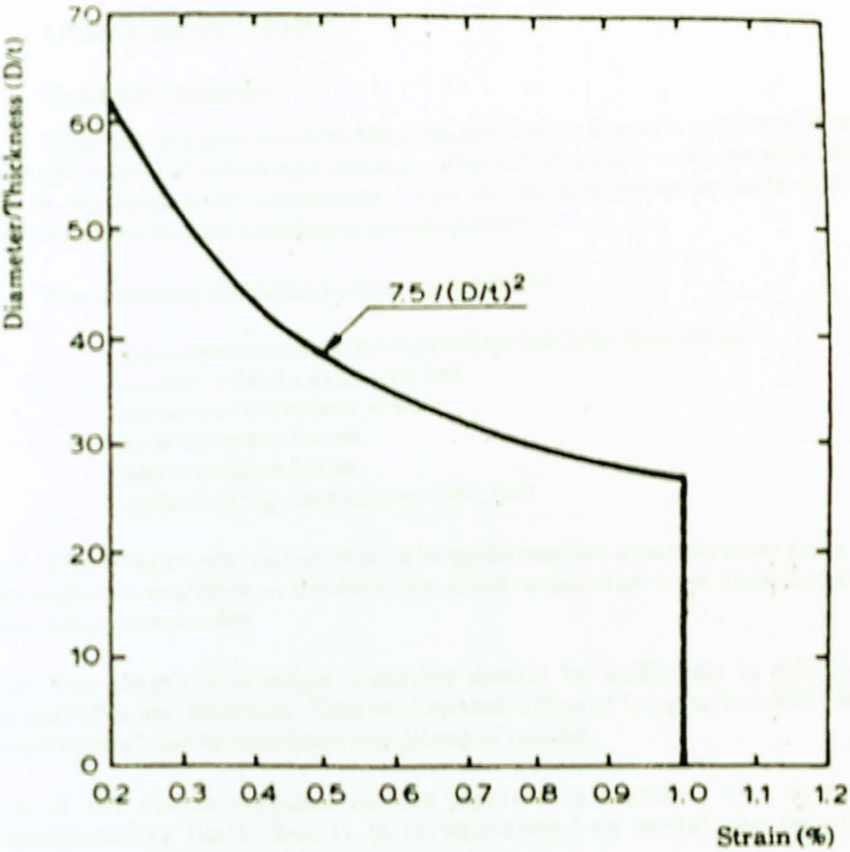


Fig. 4.1 Limiting Strain

4.3.6 If a large number of strain cycles are anticipated due to the lateral movement of the pipeline, these should be included in the fatigue assessment of the pipeline, as outlined in section 4.2.4 of /1/.

4.4 Other Relevant Criteria

4.4.1 The lateral movement of the pipeline should not result in significant damage to the external pipeline coating, as a result of abrasion from the seabed.

4.4.2 The lateral movement of the pipeline should not result in damage to sacrificial anodes attached to the pipeline.

4.4.3 The lateral movement of the pipeline should not interfere with other pipelines or other subsea installations.

5. ANALYSIS METHODS

5.1 Dynamic Analysis

5.1.1 Dynamic analysis involves the dynamic simulation of a section of pipeline under the action of waves and current. The full dynamic analysis will only be used in specialized circumstances. Due to the nonlinear behaviour of the pipeline, a time domain solution is recommended.

5.1.2 The following should be accurately modelled :

- wave spectrum and corresponding realistic time series
- current velocity at the sea bed
- structural behaviour of pipe
- hydrodynamic forces
- soil resistance forces
- restraints (eg riser connections, etc)

5.1.3 The dynamic simulation should be performed for a complete sea state. If no information is available on the duration of sea states then a sea state duration of 3 hours is recommended.

5.1.4 The length of pipeline modelled should be sufficient to adequately represent the real situation. This implies that different lengths (e.g. 250-1000 m) should be analyzed to determine sensitivity of results.

5.1.5 If the strain response of the pipeline is critical, i.e., above the proportionality limit then it is recommended to model the non-linear stress/strain behaviour of the pipe material.

5.1.6 A method of realistically representing the hydrodynamic forces experienced by the pipeline should be used. Two such methods are those presented in /10/ and /12/.

5.1.7 It is recommended that the method of modelling the soil resistance forceincludes both the effect of friction between the pipe and the soil, and the resistance due to the penetration of the pipe into the soil. One such model is that developed by Wagner et. al. /5/.

5.2 Generalized Stability Analysis

5.2.1 This method of pipeline stability analysis is based on generalization of the results from a Dynamic Analysis, through the use of a set of non-dimensional parameters and for particular end conditions.
The limitations of the method are given in section 5.2.5.
In Appendix B a calculation example is given.
The method is based on the work published in /8/ and /11/.

The major assumptions are as follows:

- hydrodynamic forces modified for wake effects
- no initial embedment
- no prior load history
- rough pipe
- passive soil resistance due to partial penetration of the pipe into the soil under cycle loading is included

- medium sand soil
- JONSWAP wave spectrum
- no reduction of hydrodynamic forces due to pipe penetration

5.2.2 The generalized response of the pipeline in a given sea state is principally controlled by the following non-dimensional parameters.

Load parameter (significant KC-number)	$K = U_s T_u / D$
Pipe weight parameter	$L = W_s / 0.5 \rho_w D U^2_s$
Current to wave velocity ratio	$M = U_C / U_s$
Relative soil weight (for sand soil)	$G = (\rho_s - \rho_w) / \rho_w = \rho_s / \rho_w - 1$
Shear strength parameter (for clay soil)	$S = W_s / (D S_u)$
Time parameter	$T = T_1 / T_u$

where:

- U_s and T_u are the near bottom significant velocity normal to the pipeline and zero up-crossing period, respectively, due to a given surface sea state.
- U_C is the steady current component in the boundary layer normal to the pipeline. An average value integrated over the diameter of the pipeline is used.
- W_s and D are the submerged pipe weight and outer diameter, respectively.
- ρ_w and ρ_s are the mass density of sea water and sand soil material, respectively.
- S_u is the undrained shear strength of a clay soil.
- T_1 is the duration of the sea state in seconds.

5.2.3 Pipeline on sand soil

For a pipeline on sand soil the generalized response is given in terms of lateral displacement for a free section and bending strain corresponding to a fixed point along the pipeline. The displacement includes the expected net displacement plus one standard deviation plus the maximum amplitude of displacement in a single wave. The Design Method determines the pipe weight that satisfies the given criteria for displacement and strain in the design sea state.

5.2.3.1 Figures 5.1 to 5.6 give the generalized weight parameter L, versus K for specific M values, solid lines. Figures are given for values of the scaled lateral pipe displacement, $\delta = Y / D$, of 10, 20 and 40 and based on sea states with 500 and 1000 wave periods, i.e. $T = T_1 / T_u = 500, 1000$. For a given pipeline with a specified design wave environment, interpolation within these figures will give the necessary submerged pipe weight to satisfy the design criteria for lateral displacement given in Section 4.2. A few iterations on the curves may be necessary to give satisfactorily accuracy in the design weight.

Net movement predictions may be sensitive to small changes in input parameters, thus sensitivity of results to each parameter should be checked.

5.2.3.2 In Fig. 5.7 is also given the generalized weight parameter L for a complete stable pipe ($\delta = 0$) on sand soil.

5.2.3.3 The bending strain in the pipe at a fixed point along the pipeline section is also found from Figures 5.1 to 5.6 (dotted lines). The engineering strain is calculated from the generalized strain under the assumption of a thinned walled pipe:

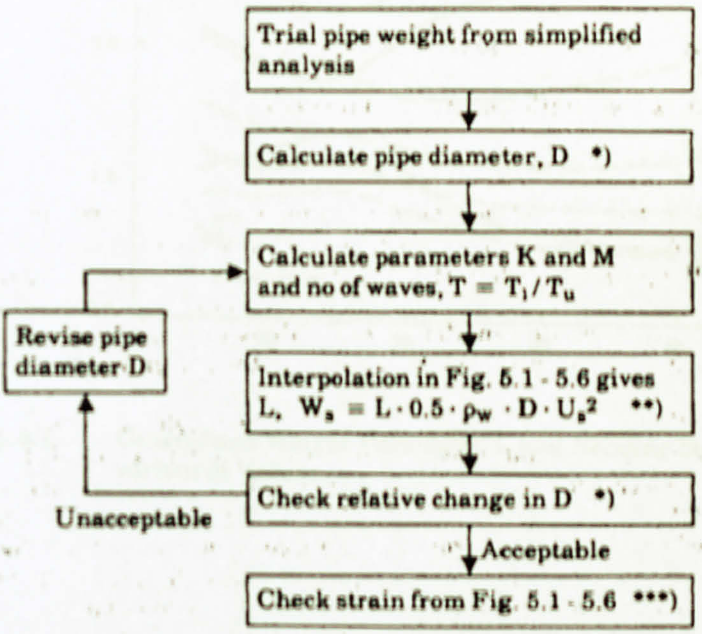
$$\varepsilon = \left(\frac{8\, W_s\, D}{\pi\, E\, t_s\, D_s} \right) + \varepsilon'$$

where

ε	-	strain
W_s	-	submerged weight
D_s	-	steel pipe diameter
t_s	-	steel pipe thickness
E	-	modulus of elasticity
D	-	pipeline outside diameter

The maximum allowed strain that can be accepted as a result from the Generalized Stability Method is 0.2 %. This is due to the use of linear elastic material properties during the development of this method. If the proportionality level (ε = 0.2 %) is exceeded a more refined analysis is recommended to study the bending strain based on a nonlinear material modelling. The use of 20 m maximum movement will generally ensure that the strain criteria is also met.

5.2.3.4 The Generalized Stability Analysis Method for sand soil is illustrated in the following flow chart:



*) $$D^2 = \frac{1}{\rho_c - \rho_w} \cdot \left[\frac{W_s}{0.25\pi g} + D_i^2 \cdot (\rho_{st} - \rho_i) + D_s^2 \cdot (\rho_{cc} - \rho_{st}) + D_{cc}^2 \cdot (\rho_c - \rho_{cc}) \right]$$

**) 5% water absorption can be assumed when calculating concrete weight

***) If strain limit (ε = 0.2 %) is exceeded

1. Increase lateral pipe weight or
2. Apply more refined analysis (non-linear material model)

Where

ρ_c	-	density of concrete coating
ρ_{cc}	-	density of corrosion coating
ρ_{st}	-	density of steel material
ρ_i	-	density of internal content
ρ_w	-	density of sea water
D_i	-	internal pipe diameter
D_s	-	outer steel pipe diameter
D_{cc}	-	outer steel pipe diameter incl. corrosion coating.

5.2.3.5 The design curves given in Figures 5.1 to 5.6 relate to a pipeline resting on a medium sand soil (ρ_s = 1860 kg/m^3). For sand soil with different density, the calculated submerged weight, W_s, should be multiplied by a correction factor according to Fig. 5.8 given as a function of the relative soil weight, $G = \rho_s / \rho_w - 1$.

Note that the curves may be used with any consistent set of units.

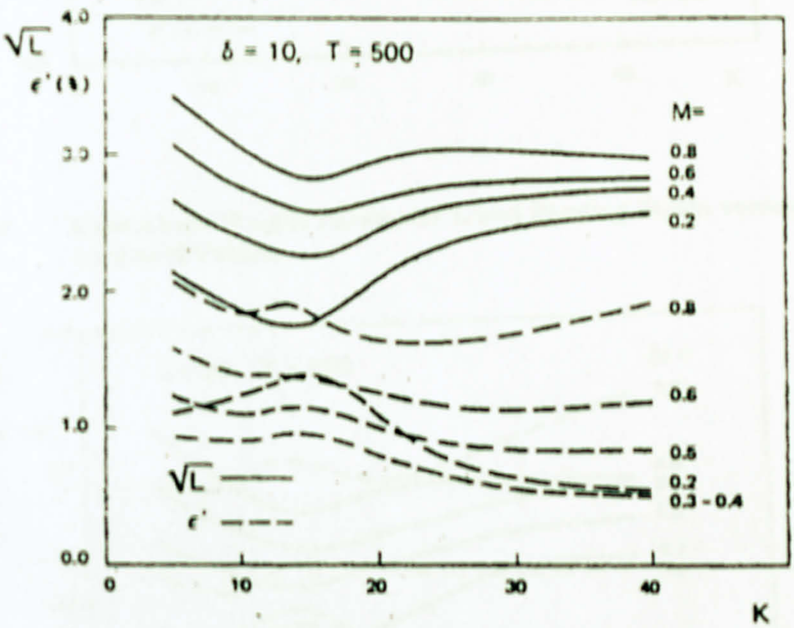


Fig. 5.1 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

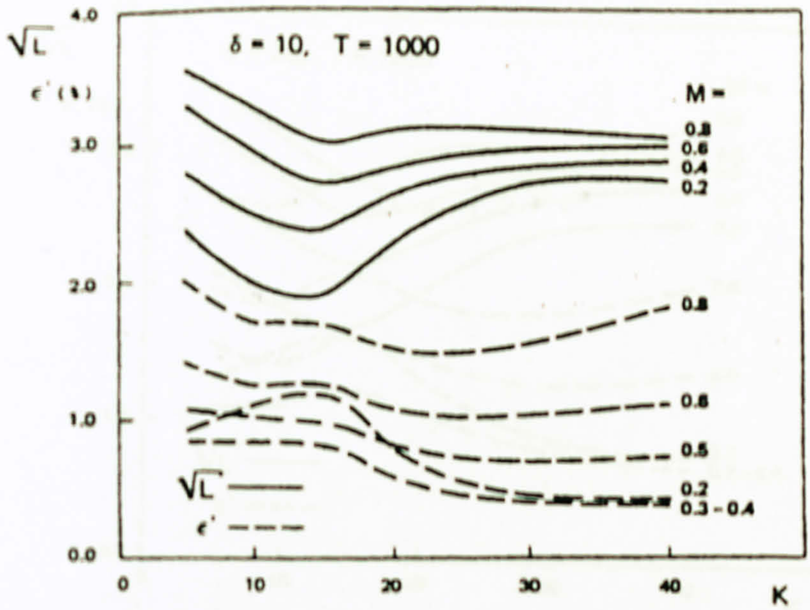


Fig. 5.2 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

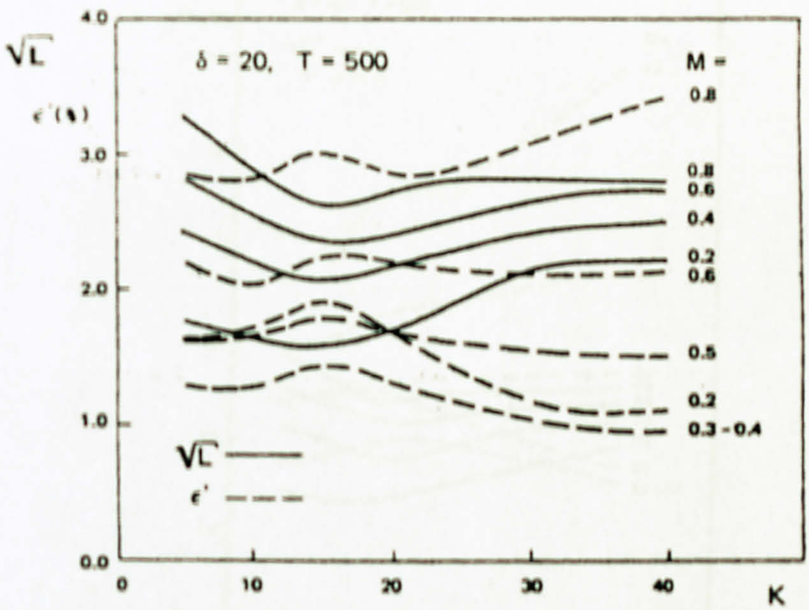


Fig. 5.3 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

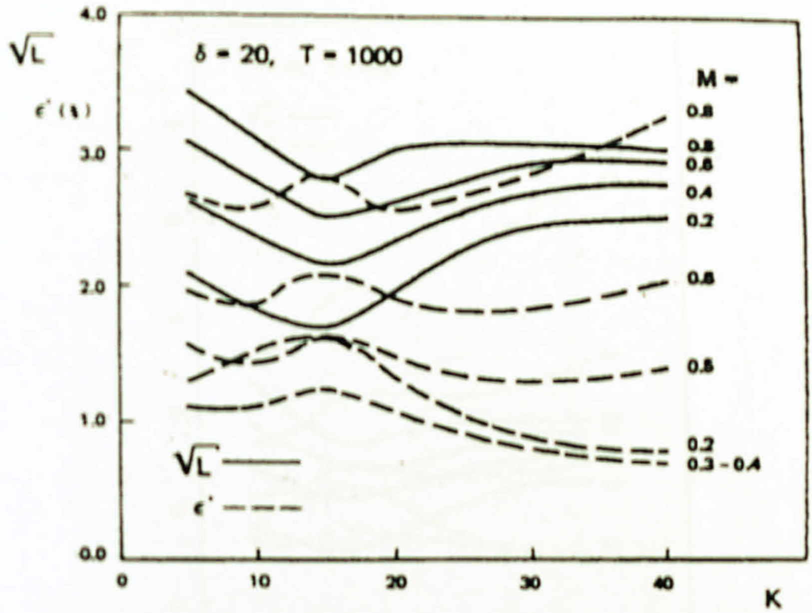


Fig. 5.4 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

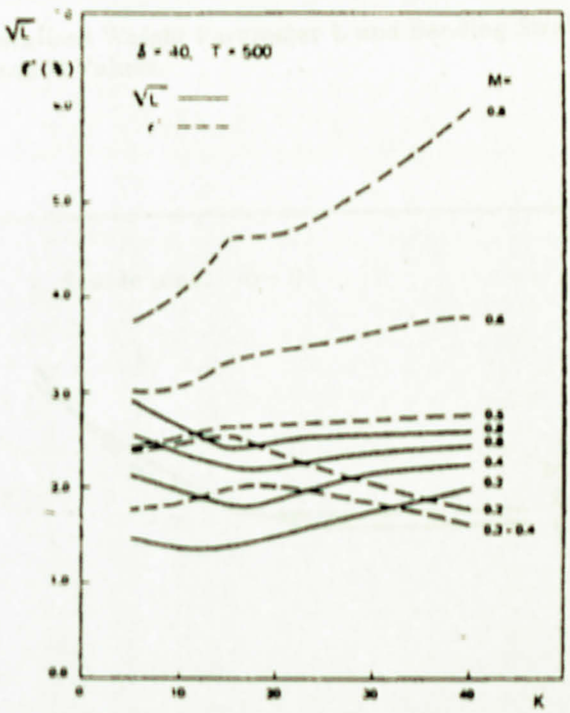


Fig. 5.5 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

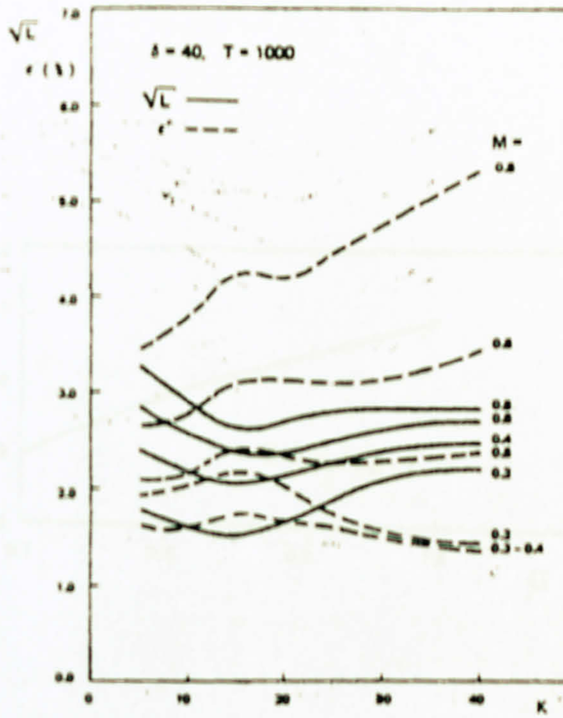


Fig. 5.6 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

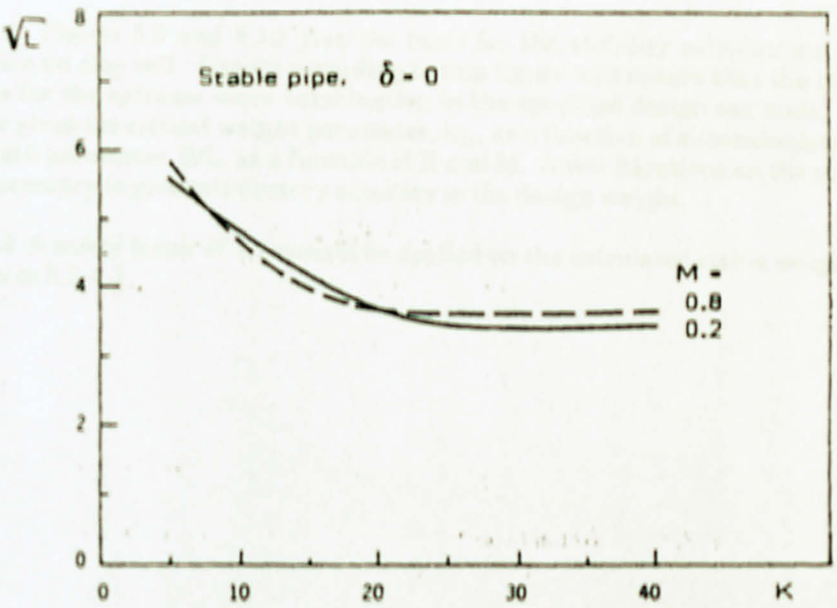


Fig. 5.7 Generalized Weight Parameter L for a Stable Pipe (δ = 0)

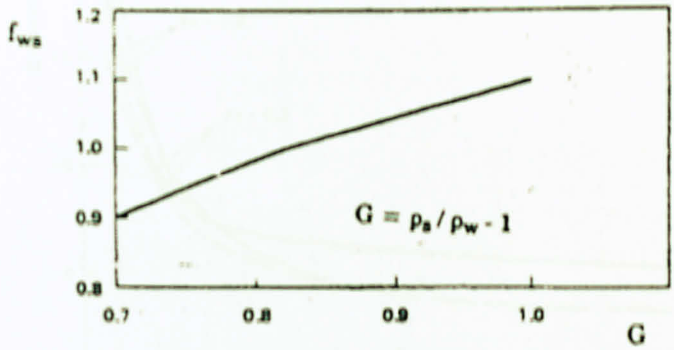


Fig. 5.8 Correction Factor on Weight W_s versus Soil Density.

5.2.4 Pipeline on clay soil

For a pipeline on clay soil the Design Method determines the pipe weight that satisfies absolute stability (no breakout) for the extreme wave in the design sea state.

5.2.4.1 Figure 5.9 and 5.10 give the basis for the stability calculations for a pipeline on clay soil. Design according to this figure will ensure that the pipe is stable for the extreme wave combination in the specified design sea state. The figure gives the critical weight parameter, L_{cr}, as a function of dimensionless soil strength parameter, S/L, as a function of K and M. A few iterations on the curves are necessary to give satisfactory accuracy in the design weight.

5.2.4.2 A safety factor of 1.1 should be applied on the calculated stable weight as shown in 5.2.4.3.

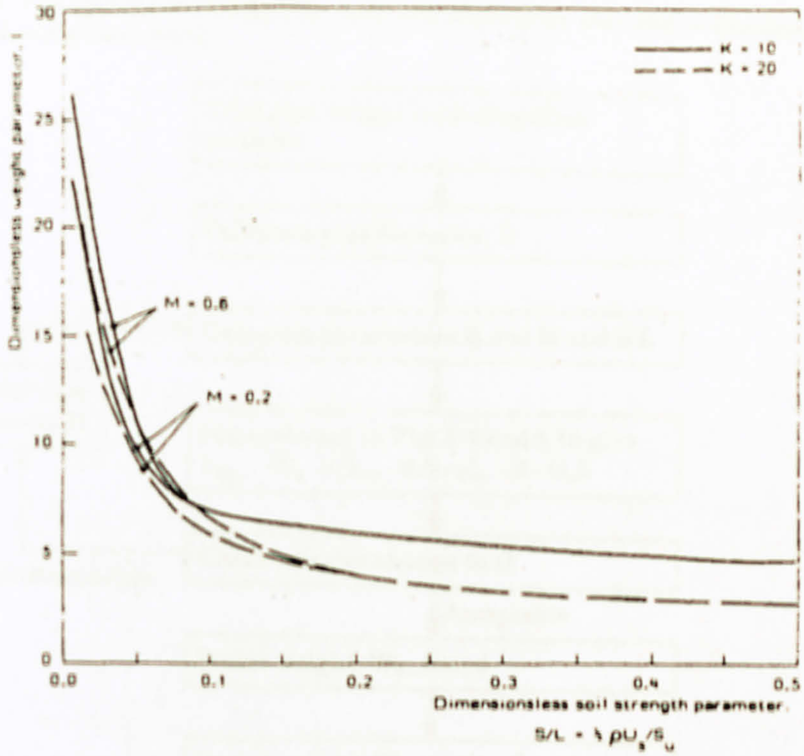


Fig. 5.9 Stability Curves for Clay (K = 10 and 20)

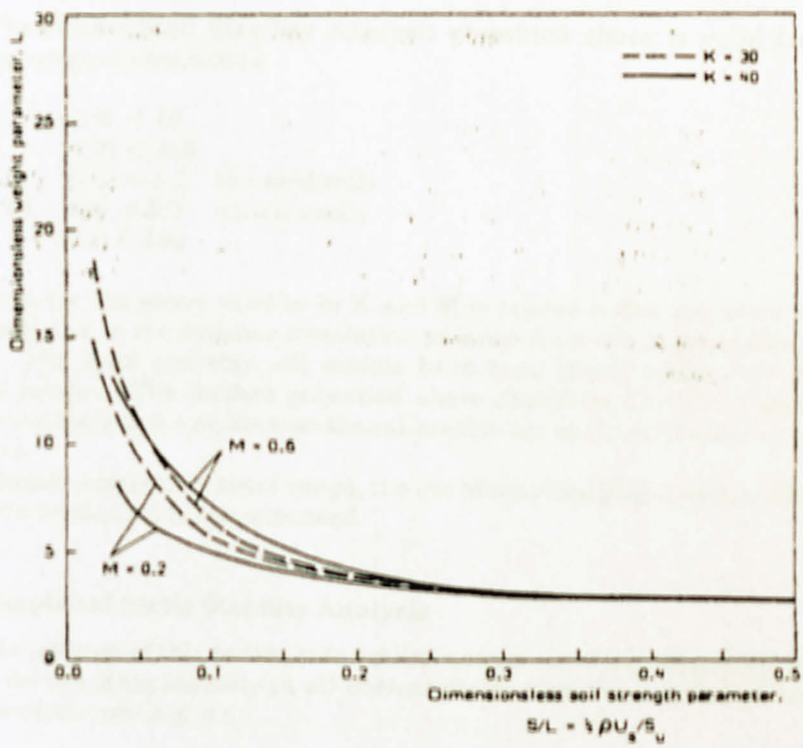
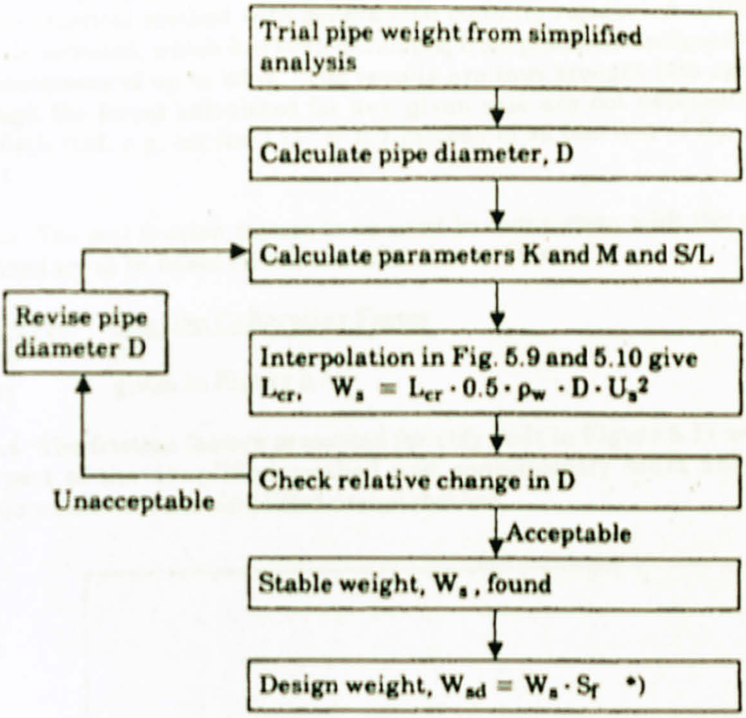


Fig. 5.10 Stability Curves for Clay (K = 30 and 40)

5.2.4.3 The Generalized Stability Analysis Method for clay soil is illustrated in the following flow chart.



*) The stable weight, W_s, calculated based on Figs. 5.9 and 5.10 must be multiplied with a safety factor S_f = 1.1 to arrive at design weight.

5.2.5 The Generalized Stability Analysis presented above is valid for the following range of parameters:

$4 < K < 40$
$0 < M < 0.8$
$0.7 < G < 1.0$ (for sand soil)
$0.05 < S < 8.0$ (for clay soil)
$D \geq 0.4$ m

The reason for the above validity in K and M is related to the use of the wake force model /10/ in the dynamic simulation program from which the method was derived. The sand and clay soil models have been tested within the above specified ranges. The method presented above should be limited to pipeline diameters (outer) ≥ 0.4 m, because the calibration has been performed for larger diameters.
For conditions outside the above range, the use of the simplified Analysis Method outlined in section 5.3 is recommended.

5.3 Simplified Static Stability Analysis

5.3.1 The purpose of this section is to outline a simple method of stability design suitable for checking stability in all normal design situations.In Appendix B a calculation example is given.

5.3.2 The method is based on a static stability approach, which ties the classical static design approach to the generalized stability method through a calibration of the classical method with generalized stability results. A calibration factor (F_w) is included, which has been developed from pipelines designed with a lateral displacement of up to 20 m. The results are thus brought into agreement even though the forces calculated for any given case are not necessarily physically realistic (ref. e.g. constant C_D = 0.7 instead of as function of R_e, K, roughness etc.).

5.3.3 The soil friction factors to be used in conjunction with the simple design method are to be based on soil classification as follows :

Soil Type	Friction Calibration Factor
Sand	0.7
Clay	given in Figure 5.11

5.3.4 The friction factors presented for clay soils in Figure 5.11 were developed as part of the simplified method and consequently must only be used in conjunction with the simplified design method.

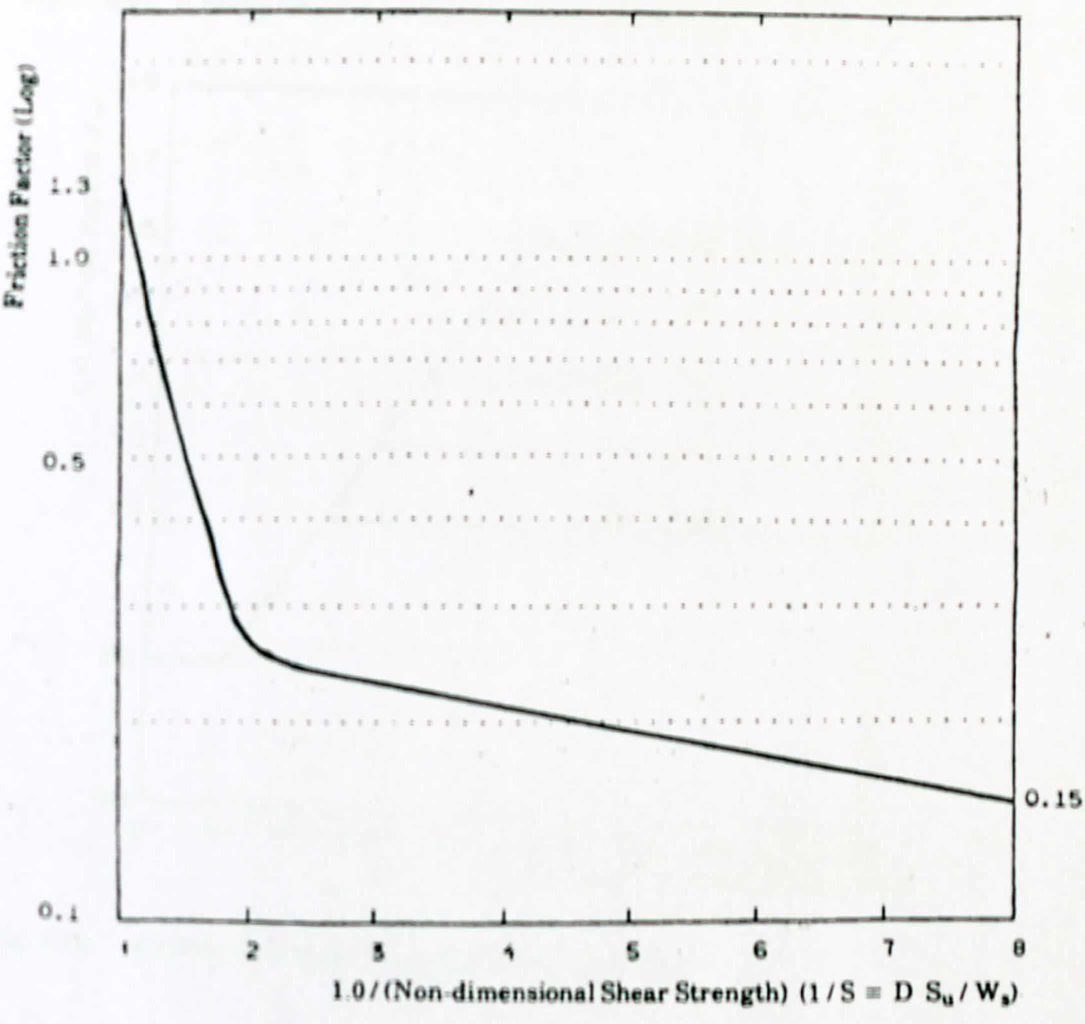


Fig. 5.11 Recommended Friction Factors for Clay (Simplified Design Method)

5.3.5 Stability in this quasi-static method is given by the following expression :

$$(W_s/F_w - F_L) \mu \geq F_D + F_I$$

where

W_s = submerged weight of the pipe
F_w = calibration factor
μ = soil friction factor
F_L = lift force
F_D = drag force
F_I = inertia force

5.3.6 The limiting value of submerged weight can then be found from :

$$W_s = \left[\frac{(F_D + F_I) + \mu \cdot F_L}{\mu}\right]_{max} \cdot F_w$$

5.3.7 The variation of the calibration factor, F_w with K and M is shown in Figure 5.12. A safety factor of 1.1 is inherent in the calibration factor F_w.

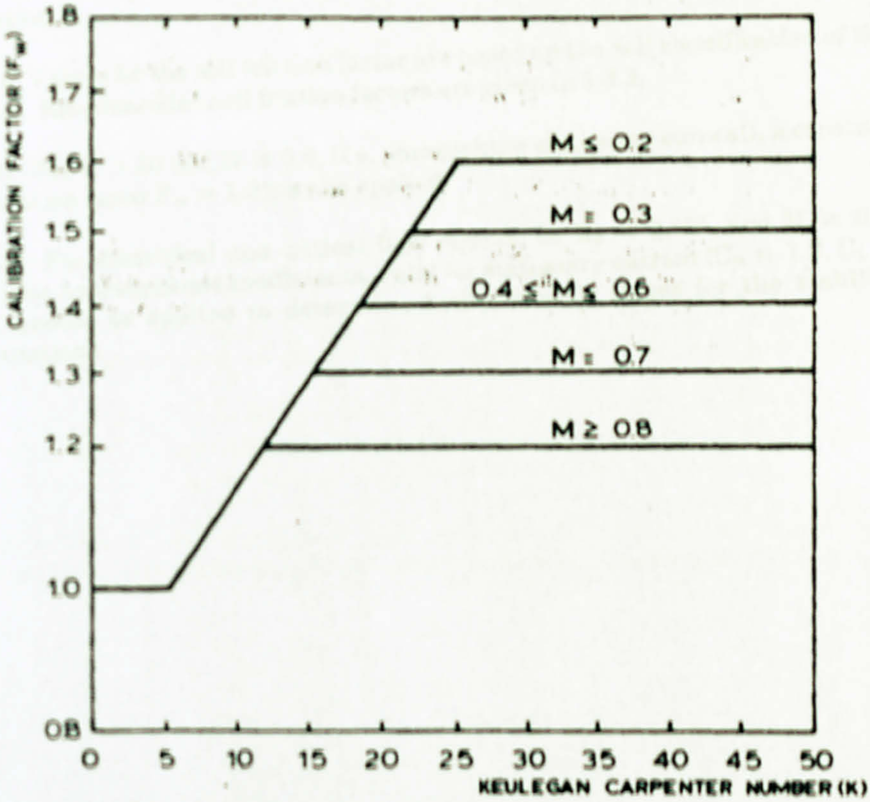


Fig. 5.12 Calibration Factor, F_w as Function of K and M

5.3.8 When using the calibration factor F_w, to calculate W_S, the hydrodynamic forces acting on the pipe (F_L, F_D and F_I) may be estimated from the following expressions:

$$F_L = \tfrac{1}{2} \cdot \rho_w \cdot D \cdot C_L \cdot (U_s \cdot \cos\theta + U_C)^2$$

$$F_D = \tfrac{1}{2} \cdot \rho_w \cdot D \cdot C_D \cdot |(U_s \cdot \cos\theta + U_C)| \, (U_s \cdot \cos\theta + U_C)$$

$$F_I = (\pi \cdot D^2)/4 \cdot \rho_w \cdot C_M \cdot A_s \cdot \sin\theta$$

where

ρ_w	=	mass density of water
D	=	total outside diameter of the pipe
C_L	=	lift force coefficient (C_L = 0.9)
C_D	=	drag force coefficient (C_D = 0.7)
C_M	=	inertia force coefficient (C_M = 3.29)
U_s	=	significant near-bottom velocity amplitude perpendicular to the pipeline
U_C	=	current velocity perpendicular to the pipeline
A_s	=	significant acceleration perpendicular to the pipeline (= $2\pi\, U_s / T_u$)
θ	=	phase angle of the hydrodynamic force in the wave cycle.

5.3.9 Information on the estimation of the water particle characteristics is given in section 2.

5.3.10 Values for the soil friction factor are based on the soil classification of the seabed. Recommended soil friction factors are given in 5.3.3.

5.3.11 For K > 50 and M ≥ 0.8, (i.e. approaching stationary current), a constant calibration factor F_w = 1.2 may be applied.

5.3.12 For subcritical and critical flow regime, i.e $R_e < 3.10^5$, and M ≥ 0.8, realistic hydrodynamic coefficients, valid for stationary current (C_D = 1.2, C_L = 0.9), should be applied to determine hydrodynamic forces for the stability calculations.

6. REFERENCES

/1/ Rules for Submarine Pipeline Systems, Det norske Veritas, 1981

/2/ Wolfram Jr., W.R., Getz, J.R. and Verley, R.L.P.: "PIPESTAB Project: Improved Design Basis for Submarine Pipeline Stability", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5501, Houston, 1987.

/3/ Holthe, K., Sotberg, T. and Chao, J.C.: "An Efficient Computer Model for Predicting Submarine Pipeline Response to Waves and Current", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5502, Houston, 1987.

/4/ Verley, R., Lambrakos, K.F. and Reed, K.: "Prediction of Hydrodynamic Forces on Seabed Pipelines", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5503, Houston, 1987.

/5/ Wagner, D., Murff, D., Brennodden, H. and Sveggen, O.: "Pipe-Soil Interaction Model", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5504, Houston, 1987.

/6/ Slaattelid, O.H., Myrhaug, D. and Lambrakos, K.F.: "North Sea Bottom Boundary Layer Study for Pipelines", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5505, Houston, 1987.

/7/ Tryggestad, S., Kollstad, T., Nilsen, J. and Selanger, K.: "Measuring Technique and Field Data for Pipeline Stability Studies", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5506, Houston, 1987.

/8/ Lambrakos, K.F., Remseth, S., Sotberg, T. and Verley, R.: "Generalized Response of Marine Pipelines", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5507, Houston, 1987.

/9/ RP D301: Site Investigation for Offshore Structures, Veritec, 1987.

/10/ Lambrakos, K.F., Chao, J.C., Beckman, H. and Brannon, H.R. "Wake Model of Hydrodynamic Forces on Pipelines," Ocean Engineering, Vol. 14, No. 2, pp 117-137

/11/ Sotberg T., Lambrakos, K.F., Remseth, S., Verley, R.L.P., and Wolfram, W.R., "Stability Design of Marine Pipelines", BOSS' 1988.

/12/ Jacobsen, V., Bryndum, M.B., and Tsahalis, D.T., "Prediction of irregular wave forces on submarine pipelines". Proc. 7th OMAE, Houston, Febr. 1988.

APPENDIX A

APPROXIMATE METHOD TO CALCULATE BOUNDARY LAYER REDUCTION

A.1 INTRODUCTION

This Appendix presents an approximate method for calculating a boundary layer reduction factor which may be applied to the steady current velocity used in the calculation of pipeline stability.

The method can be applied to both steady current and combined wave/steady current flow conditions over fixed bottoms of granular materials. The effect of the seabed roughness and wave/current interaction are accounted for in this simplified procedure. However other effects such as sediment transport and ripple formation are not included. These effects will in general lead to greater velocity reductions.

The calculation procedure outlined below is based on work reported in /A1/.

A.2 VELOCITY PROFILE

The steady flow is described as a logarithmic velocity profile of the form:

$$U(z) = \frac{U^*}{\kappa} \ln\left[\frac{z+z_o}{z_o}\right] \tag{A.1}$$

where

U^* = friction velocity
κ = von Karman's constant (= 0.4)
z = elevation above the seabed
z_o = bottom roughness parameter

The average steady velocity acting over the pipe is appropriate for use in determining the hydrodynamic forces on the pipe.

The average velocity acting over a pipe of diameter D, is given by :

$$U_D = \frac{1}{D}\int_o^D U(z)\,d(z) \tag{A.2}$$

The ratio between this average velocity and a known reference steady velocity, U_r, at some height z_r above the seabed is given by :

$$\frac{U_D}{U_r} = \frac{1}{ln(z_r/z_o+1)} \cdot \frac{1}{D} \cdot \int_o^D ln\left[\frac{z+z_o}{z_o}\right]dz$$

$$= \frac{1}{ln(z_r/z_o+1)} \cdot \left\{\left[1+z_o/D\right]ln\left[D/z_o+1\right]-1\right\} \tag{A.3}$$

z_r may be taken as 3 m if no other information is available.

A.3 CURRENT FLOW

For the case of steady current flow acting alone the effect of the seabed roughness (grain size) may be accounted for in the boundary layer velocity estimation.

The mean grain size, d_{50}, is related to Nikuradse's equivalent sand roughness parameter, K_b, and to the bottom roughness by :

$$K_b = 2.5\, d_{50} \tag{A.4}$$

$$z_o = \frac{K_b}{30}$$

The mean grain size may be estimated from Table A.1.

The following procedure may then be followed to estimate the boundary layer reduction.

/1/ Estimate the mean grain size (d_{50}) from soil samples or from Table A.1

/2/ Calculate K_b, z_o, D/z_o and z_r/z_o

/3/ Calculate the velocity reduction factor, U_D/U_r from equation (A.3)

A.4 COMBINED WAVE AND CURRENT FLOW

The non-linear interaction between the wave and the current flow results in a modification of the steady velocity profile. This modification of the steady flow component is attributed to an apparent increase in the seabed roughness.

The apparent roughness (z_{oa}) is dependant on the ratio between the wave induced velocity and the steady current velocity, given by :

$$U_s/U_r$$

where

U_s = significant horizontal wave induced velocity at the reference distance (z_r) above the seabed.

The apparent roughness is also dependant on the relative roughness parameter given by :

$$A_o/K_b$$

where

A_o = orbital semi-diameter of the water particles associated with U_s, i.e. $A_o = (U_s T_p / 2\pi)$

The determination of the boundary layer reduction factor is based on similar assumptions to the steady flow case. In addition it is assummed that the bottom material does not form into ripples, and that the steady current and the wave flow are co-directional. These are both conservative assumptions. The apparent roughness (z_{oa}) can be obtained from Figures A.1 to A.7 and the boundary layer reduction factor then obtained from equation (A.3), with z_{oa} substituted for z_o.

This method is valid provided the following are satisfied:

$$z_r > 0.2\,A_o \left[\frac{A_o}{K_b} \right]^{-0.25}$$

$$\frac{A_o}{K_b} \geq 30$$

$$\frac{U_s}{U_r} \geq 1$$

The following procedure may be adopted to estimate the boundary layer reduction factor for combined wave and current flows :

1. Estimate the mean grain size (d_{50}) from soil samples or from Table A.1

2. Calculate K_b, z_0, z_r/K_b and K_b/A_o

3. Check that the parameters Z_r, A_o/K_b and U_s/U_r are within the ranges of validity.

4. From Figures A.1 to A.5 determine the appropriate value of z_{oa}/z_o and hence z_{oa}. This may require interpolation between figures for various values of z_r/K_b.

5. Calculate the velocity reduction factor, U_D/U_r from equation (A.3) substituting z_{oa} for z_o.

Table A.1 Grain size for seabed materials

Seabed	Grain Size d_{50} (mm)	Roughness z_0 (m)
Silt	0,0625	5.21 E-6
V. Fine Sand	0,125	1.04 E-5
Fine Sand	0,25	2.08 E-5
Medium Sand	0,5	4.17 E-5
Coarse Sand	1,0	8.33 E-5
V. Coarse Sand	2,0	1.67 E-4
Gravel	4,0	3.33 E-4
Pebble	10,0	8.33 E-4
	25,0	2.08 E-3
	50,0	4.17 E-3
Cobble	100,0	8.33 E-3
	250,0	2.08 E-2
Boulder	500,0	4.17 E-2

A.5 EXAMPLES

/1/ Current Flow

A pipeline with an external diameter of 0.5m is to be placed in a tidal stream with a velocity of 1m/s measured at 5m above the seabed. The seabed material is coarse sand. Find the average velocity acting across the diameter of the pipeline.

From the problem formulation the following are given:

$D = 0.5m$
$U_r = 1m/s$
$z_r = 5m$

For coarse sand the following can be extracted from Table A.1:

$d_{50} = 1mm$
$z_0 = 8.33\ E\text{-}5\ m$

This gives:

$D/z_0 = 6000$
$z_r/z_0 = 60000$

and substituting into equation (A.3) gives :

$U_D/U_r = 0.7$ giving $U_D = 0.7m/s$

The average velocity across the pipe diameter is 0.7m/s

/2/ Combined Wave and Current Flow

A pipeline with an external diameter of 0.5m is to placed on the seabed with a water depth of 30m. The design wave conditions for the area show a significant wave height of 8m with a peak period of 13s. The design current velocity is 1m/s measured at 5m above the seabed. The seabed material is coarse sand.
Find the average steady velocity acting on the pipe .

From the problem formulation the following are given :

$D = 0.5m$
$U_r = 1m/s$
$z_r = 5m$
$H_s = 8m$
$T_p = 13s$
$d = 30m$
$R = 1.0$

Calculating the near-bed significant wave induced particle velocity and associated period at the seabed from Figs. 2.1 and 2.2.

$U_s^* = 1.55m/s$ $T_u = 12.35s$

The amplitude of the horizontal water particle displacement is estimated as:

$$A_o = \frac{U_s^* T_u}{2\pi} = 3.05 \text{ m}$$

For coarse sand the following can be extracted from Table A.1:

d_{50} = 1mm
K_b = 2.5 E-3 m
z_o = 8.33 E-5 m

giving

$z_r / K_b = 2000$

$A_o / K_b = 1220$

Checking the regions of validity :

$$0.2A_o\left[\frac{A_o}{K_b}\right]^{-0.25} = 1.03\,E-1 \qquad \text{OK}$$

$A_o / K_b = 1220$ OK

$U_s / U_r = 1.55$ OK

From Figure A.4 for $Z_r / K_b = 2000$

$z_{oa} / z_o = 17.5$ giving z_{oa} = 1.46 E-3 m.

The velocity reduction factor is then found from equation (A.3), giving :

$U_D / U_r = 0.6$ and thus U_D = 0.6m/s

The average steady velocity across the pipe is then 0.6m/s

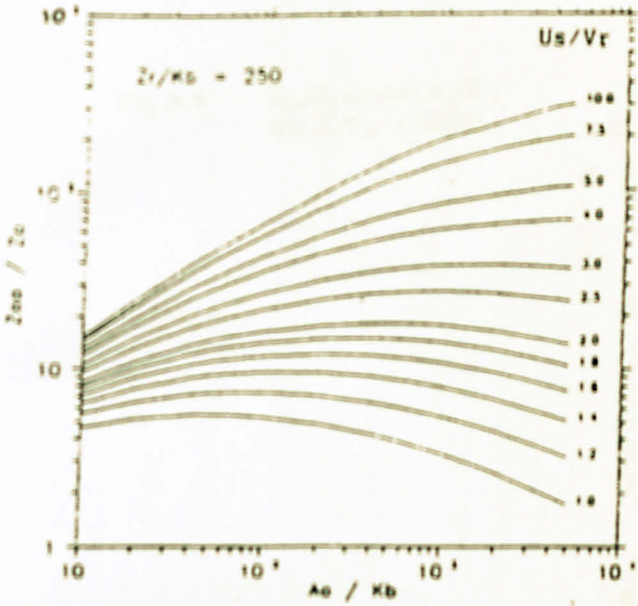


Fig A.1 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 250$)

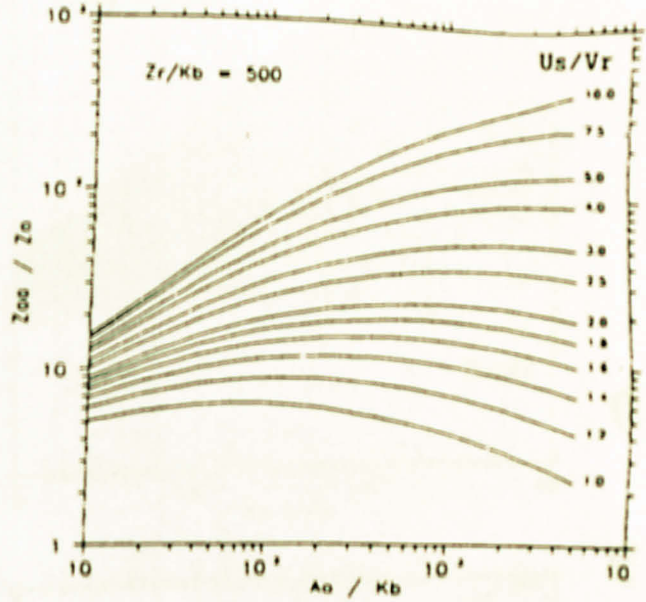


Fig A.2 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 500$)

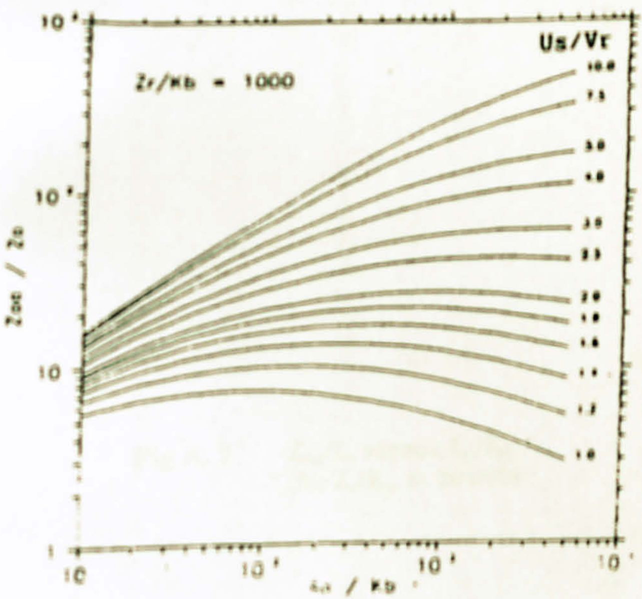


Fig A.3 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 1000$)

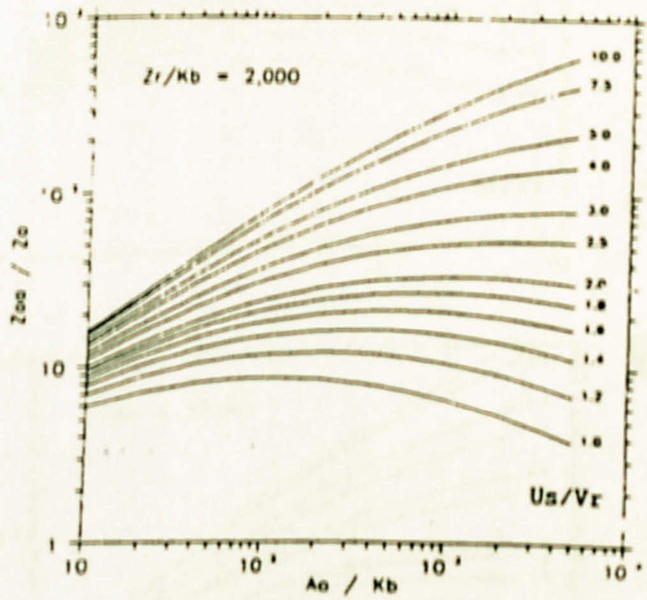


Fig A.4 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 2000$)

Fig A.5 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 5000$)

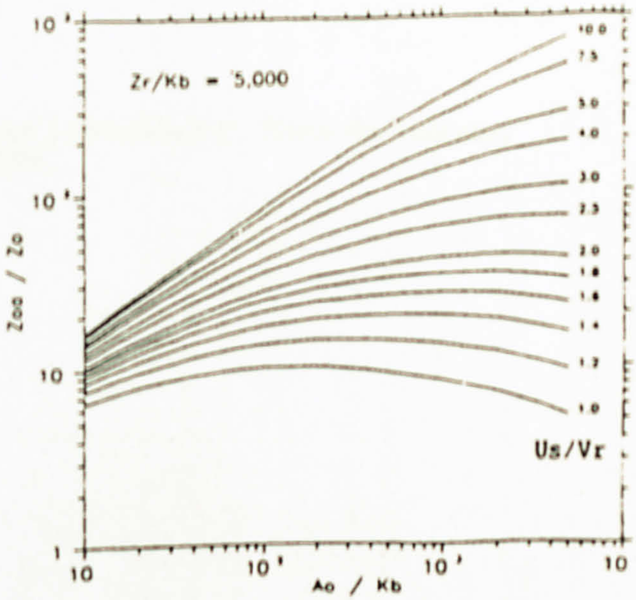


Fig A.6 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 10\,000$)

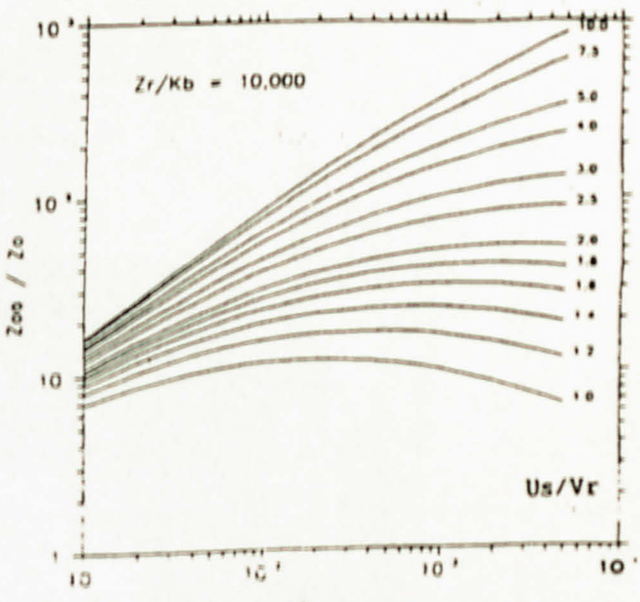
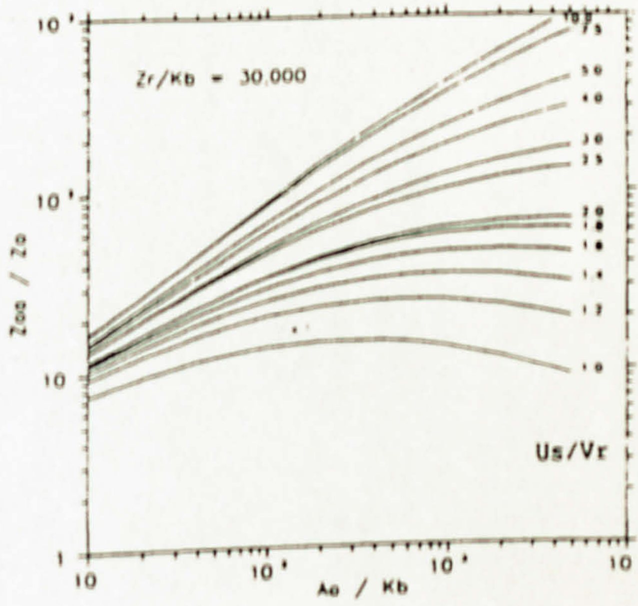


Fig A. 7 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 30\,000$)



A.6 REFERENCES

/A1/ Slaattelid,O.H., Myrhaug,D. and Lambrakos,K.F. North Sea boundary layer study for pipelines, OTC 5505, 1987.

APPENDIX B

CALCULATION EXAMPLES

B.1 INTRODUCTION

This Appendix presents some calculation examples of the simplified and generalized methods. The examples are for the following design case:

Pipeline design parameters:

- Steel pipe outer diameter, $D_s = 0.4064$ m
- Wall thickness, $t_s = 0.0127$ m
- Internal diameter, $D_i = 0.3810$ m
- Corrosion coating thickness, $t_{cc} = 0.005$ m
- Density of corrosion coating, $\rho_{cc} = 1300$ kg/m^3
- Density of concrete coating, $\rho_c = 2400$ kg/m^3
- Density of internal content, $\rho_i = 10$ kg/m^3 (gas)
- Density of seawater, $\rho_w = 1025$ kg/m^3
- Density of steel, $\rho_{st} = 7850$ kg/m^3

Soil type: Medium sand of density, $\rho_s = 1860$ kg/m^3

Environmental data:

- significant wave height, $H_s = 14.5$ m
- spectral peak period, $T_p = 15$ s
- water depth, $d = 110$ m
- current 3 m above bottom, $U_r = 0.6$ m/s

B.2 SIMPLIFIED METHOD

1. Find water particle velocities:

For wave, using Fig. 2.1 - 2.3.

$$T_n = \sqrt{(d/g)} = \sqrt{(110/9.81)} = 3.348$$

$$T_n/T_p = 3.348/15 = 0.223$$

From graph, Fig. 2.1 (Pierson Moskovitz, PM): $(U_s{}^*T_n)/H_s = 0.14$

$$U_s{}^* = (H_s/T_n)\cdot 0.14 = (14.5/3.348)\cdot 0.14 = \underline{0.606 \text{ m/s}}$$

Zero-up-crossing period, T_u - using Fig. 2.2

$$T_u/T_p = 1.07 \rightarrow T_u = 1.06 \cdot T_p = 16.05 \text{ sec.}$$

Directional and spreading factor assumed to be

$R = 1.0$ - no reduction.

$\underline{U_s = U_s{}^* \cdot R = 0.606 \text{ m/s}}$

$\underline{T_u = 16.05 \text{ sec.}}$

Current velocity:

The current velocity 3 m above seabed (Z_r = 3).

$$U_r = 0.6 \text{ m/s}$$

To calculate average velocity across the pipe assuming an approximate pipe diameter of 0.5 m (i.e. including corrosion coating plus 40 mm of concrete coating).

Medium sand assumed, from Table A1,

$$d_{50} = 0.5 \text{ mm}$$
$$Z_0 = 4.17 \cdot 10^{-5} \text{ m}$$

which gives:

$$D / Z_0 = 11990$$

$$Z_r / Z_0 = 3.0 / 4.17 \cdot 10^{-5} = 71942$$

Substituting in equation A.3:

$$\frac{U_D}{U_r} = \frac{1}{\ln(71942+1)} \cdot \left\{ \left[1 + \frac{1}{11990} \right] \ln(11990+1) - 1 \right\}$$

$$U_D / U_r = 0.7504$$

$$U_D = 0.7504 \cdot U_r = 0.6 \cdot 0.7504 = 0.45 \text{ m/s}$$

2. Using simplified static stability method:

Medium sand has been assumed, μ = 0.7.

$$C_L = 0.9, C_D = 0.7, C_M = 3.29$$

An approximate diameter, D ≈ 0.5 m

$$A_s = 2\pi \cdot \frac{U_s}{T_u} = 2\pi \cdot \frac{0.606}{16.05} = 0.2372 \text{ m/s}^2$$

$$M = \frac{U_D}{U_s} = \frac{0.45}{0.606} = 0.75$$

$$K = \frac{U_s \cdot T_p}{D} = \frac{0.606 \cdot 16.05}{0.5} = 19.45$$

From Fig. 5.12, F_w = 1.25

Computing hydrodynamic forces and iterating to find the phase angle (θ) giving maximum submerged weight requirement (W_s).

For θ = 21 degrees, max W_s is found:

$$\left.\begin{array}{l} F_L = 237.9\ \text{N/m} \\ F_D = 185.1\ \text{N/m} \\ F_I = 56.4\ \text{N/m} \end{array}\right\} W_s = \left[\frac{(185.1 + 56.4) + 0.7 \cdot 237.9}{0.7}\right] \cdot 1.25\ [\text{N/m}]$$

$$W_s = 728.75\ \text{N/m}$$

<u>A minimum submerged weight of 728.75 N/m is required.</u>

(Calculate concrete density required to achieve the above submerged weight with the estimated concrete thickness. Revise concrete thickness and density as necessary and repeat until a satisfactory combination of density and thickness is achieved).

B.3 GENERALIZED METHOD

From simplified static analysis, we have determined the following start values:

W_s = 728.75 N/m
D = 0.5 m (initial approximate outer pipe diameter)

Using the flowchart, section 5.2.3.4, assume thicknesses in first trial to be as for Simplified Method above.

Check diameter against formula:

$$D = \left\{\frac{1}{2400 - 1025}\left[\frac{728.75}{0.25 \cdot \pi \cdot 9.81} + 0.3810^2(7850 - 10) + 0.4064^2(1300 - 7850) + 0.4184^2(2400 - 1300)\right]\right\}^{\frac{1}{2}} [\text{m}]$$

D = 0.5 m → required outer diameter.

Calculate parameters: (environmental data from simplified static stability method).

$$K = \frac{U_s \cdot T_u}{D} = \frac{0.606 \cdot 16.05}{0.5} = 19.45$$

$$M = \frac{U_D}{U_s} = \frac{0.45}{0.606} = 0.75$$

$$T = \frac{T_1}{T_u} = \frac{3 \cdot 60 \cdot 60}{16.05} = 672.90 \text{ (3 hours storm duration)}$$

$$\text{Target displacement} = 10\ \text{m};\ \delta = \frac{\text{displacement}}{\text{D}} = \frac{10}{0.5} = 20$$

Using Fig. 5.1 to 5.6 to determine L by interpolating with respect to values for δ and T as necessary:

$$\left.\begin{array}{l} \delta = 20,\ T = 500 \text{ give } \sqrt{L} = 2.65 \\ \delta = 20,\ T = 1000 \text{ give } \sqrt{L} = 2.85 \end{array}\right\} \text{interpolating, } \sqrt{L} = 2.72$$

$$\rightarrow L = 7.40$$

Computing new

$$\begin{aligned} W_s &= L \cdot 0.5 \cdot \rho_w \cdot D \cdot U_s^2 \\ &= 7.40 \cdot 0.5 \cdot 1025 \cdot 0.500 \cdot 0.606^2\ \text{N/m} \\ W_s &= 696.4\ \text{N/m} \end{aligned}$$

Compute new D:

$$D = \left\{ \frac{1}{2400 - 1025} \left[\frac{696.4}{0.25 \cdot \pi \cdot 9.81} + 0.3810^2 (7850 - 10) + 0.4064^2 (1300 - 7850) + 0.4184^2 (2400 - 1300) \right] \right\}^{\frac{1}{2}} [\text{m}]$$

D = 0.497 m (i.e. 0.6% difference from trial figure of 0.500 m, therefore acceptable).

Check strain level:

From Fig. 5.1 - 5.4, by interpolation $\varepsilon' = 2.6\%$

Engineering strain, section 5.2.3.3:

$$\varepsilon = \left(\frac{8 \cdot 666.3 \cdot 0.500}{\pi \cdot 2.1 \cdot 10^{11} \cdot 0.0127 \cdot 0.4064} \right)^{\frac{1}{2}} \cdot 2.6 = 0.0023\ \%\ :\ \text{OK (i.e.} < 0.2\ \%)$$

Distribution
Veritas Offshore Technology and Services A/S
The Library
Veritasveien 1
P.O.Box 300
N-1322 Høvik, Norway
Telex: 76 192 verit n
Telephone: +47 2 47 72 50
Facsimile: +47 2 47 74 74
Telegraphic address: Veritas, Oslo

Printed and published by Veritec.